SPLICES IN TENSILE REINFORCING BARS

by

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A RESEARCH REPORT

"SPLICES IN TENSILE REINFORCING BARS"

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PART I : Static Tests of Splices

PART II : Flexural Beam Tests

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Research assistants serving on the project were Richard L. Gilbert, Ronald D. Wickens, and Robert P. Williams, all students of civil engineering at the University of Oklahoma Research Institute.

SYNOPSIS

The purpose of this project was to provide information on the relative merits of the various methods of connecting tensile reinforcing bars in reinforced concrete construction.

This project consisted of two parts. In Part I a series of 122 different bar samples were prepared and tested to failure. The deformed bar sizes considered were Nos. 9, 10, 11, and 14S for material having a nominal yield strength of 40,000 psi (ASTM-A15 and ASTM-A408) and a nominal yield strength of 60,000 psi (ASTM-A432). A total of eight different splicing systems were studied plus a series of control bars (continuous bars without any splices). Stress-strain diagrams are included for all samples that yielded sufficient data. On the basis of this study, the most promising types of splicing devices were selected for flexural tests which constituted Part II of this project.

In Part II, a total of eight reinforced concrete beams were formed, having a single No. 11 size, (ASTM-A15) deformed reinforcement bar in the bottom of each member.

Each beam was twelve feet in length and had a cross-section of 8" wide by 12" deep. A total of four different splicing devices were studied in this series plus one beam which contained a similar, continuous reinforcing bar (without a splice) for control purposes.

The results of Part I and Part II of this project are reported herein.

PART I

STATIC TESTS
of
SPLICED REINFORCING BARS

CHAPTER I. INTRODUCTION

Object of this research

The need for reliable, practical and economical methods of splicing tensile reinforcing bars has long been known. In the field of precast concrete, such a connection or splice would lead to continuity for precast members resulting in increased strength, rigidity, and economy. In many long span, flexural and eccentrically-loaded compression members it is frequently necessary to splice tensile reinforcing bars.

In the past this problem has been taken care of by lapping or butt-welding individual bars. Both of these approaches can adequately transfer the axial stress, but each has inherent undesirable characteristics. In the case of lapping the bars, it usually initiates diagonal tensile cracks or failure at the point of cutoff of the stressed bars unless extra stirrups or ties are supplied in this region. The butt-welding of the ends of reinforcing bars is generally considered to be too expensive to be a practical solution. Because of these undesirable features, other suitable means of tensile splicing would have wide acceptance provided proper design criteria could be developed through research.

Because of this design need, several mechanical connectors have appeared on the market. However, due to the absence of proper research and testing, only limited acceptance has been gained. This is evidenced by the fact that most codes do not recognize such splices. In the 1961 American Association of State Highway Officials (AASHO) Specifications (Sec. 1.7.5(c)) the following statements appear: "Tensile reinforcement shall <u>not</u> be spliced at points of maximum stress. When the reinforcement is spliced, the spliced bars shall lap sufficiently to develop the full strength in bond." A further statement appears in Section 2.5.6: "Splicing of bars, except where shown on the plans, will not be permitted without the written approval of the engineer. Splices shall be staggered as far as possible. Unless otherwise shown on the plans, bars in the bottom of beams and girders, and in walls, columns and haunches shall be lapped 20 diameters and bars near the top of beams and girders having more than 12 inches of concrete under the bars shall be lapped 35 diameters, to make the splice." The 1963 Code of the American Concrete Institute (ACI) does not recommend the splicing of tensile bars at points of maximum stress, but such a splice when used is required to develop the full computed stress in the bar based on not more than 3/4 of the permissible bond value. In addition a minimum lap is specified for each of several grades of steel bars. To meet the 1963 ACI code, a welded butt splice or other approved positive connection must develop in tension at least 125% of the specified yield strength of the bar.

If mechanical splices are to have general acceptance, comparative data must be obtained on the relative merits of mechanical splices as compared to lap or butt welded splices. The collection of this data and the critical evaluation of some of these types of connections were the objectives of this research.

Present status of research

In 1959 Eriksson (5)* studied the sleeve splice. Eriksson's testing program of 200 samples included only static tests, but treated extensively the variables of sleeve configuration, curing time, grout strength, grout type, and grout thickness. Both tension and compression tests were performed. These tests showed the splices to have excellent reliability under compression loading and to react

*Numbers refer to reference shown in the list of "Selected References" at the end of Part I.

in a satisfactory manner to tension loadings. Eriksson's tests indicated that the splice was quite insensitive to the variables of mortar thickness, grout type and strength, and curing durations above seven days, but highly sensitive to the splice sleeve length in the range of two to ten bar diameters. Pending a long-term study on the effect of shrinkage on the "plain" grout splice , Eriksson recommended the use of nonshrinking mortar grout.

Other references studied in the literature were those dealing with lapped splices and with bond stresses. Although lapped splices have long been an accepted construction practice, the literature in this area was found to be rather limited. The problems of bar spacing, type of bar deformations and lap length were investigated by Chamberlin (2), Walker (9), and Chinn, Ferguson and Thompson (3).

Kluge and Tuma (7) conducted tests on lapped splices in beams. The largest bar tested was a #8. Calculations based on this data indicated that the relative movement of points at the ends of the lapped splice was 0.008 inches at 18,000 psi and 0.020 inches at 40,000 psi. There is very extensive literature on the subject of bond stresses and tension pull-out specimens. One of the earliest in this area is by Abrams (1). Abrams developed sound ideas on the nature of bond resistance and conducted extensive tests on pull-out and beam specimens. He illustrated the progressive nature of bond stress development and described in detail the effect of bar deformations on maximum bond stresses.

Since the paper by Abrams a number of investigators have conducted tests on the bond resistance of various types and sizes of reinforcing bars. Some of the most significant of these papers are those by Watstein (10), Clark (4), Mylrea (8), and Ferguson, Turpin, and Thompson (6).

Scope of this investigation

Part I was divided into two main groups: one of 40 ksi nominal yield strength bars (ASTM-A15 and ASTM-A408) and one of 60 ksi (ASTM-A432) nominal yield strength deformed bars. All bars used during project were from the same mill, and thus had the same deformation pattern. A series of control bars (continuous bars) and a series of each type connection were tested using #9, #10, #11 and #14S bars in each group. The control bars were of the same heat number as the spliced bars of the same size and yield strength. Thus any variation in performance between the control bar and each type of the connection could reasonably be attributed to the connection alone. Two control samples of each particular size and yield strength were tested. The results of these two tests were also compared to the mill test reports which were supplied by the fabricator.

Two samples of each bar size and steel grade of the butt welded bars were tested. These results then were compared to the other splicing devices.

After some study it was decided to limit the new type connections to three basic types and to designate them by their general characteristics; namely, exothermic #1, exothermic #2 and sleeve-with-metal-filler. Three identical samples of each bar size and steel grade were included.

Sleeves filled with epoxy and sleeves filled with expanded grout were also included in this investigation. Since it was felt that these connections would have to compare favorably to the sleeve-with-metal-filler connection to have significant practical value, they utilized the same sleeve that was used in the sleeve-with-metal-filler connection of the same bar size and steel grade. All bars were deformed #9 bars of 40 ksi nominal yield point (ASTM-A15). Five sleeve samples composed of expanded grout and at least three samples of each of three epoxy formulations were tested.

The suppliers of the splices and material used in this project were as follows:

SLEEVE-WITH-METAL-FILLER

"Cadweld" Splice by
Erico Products, Inc.
2070 E 61st Place
Cleveland 3, Ohio.

EXOTHERMIC #1

"Exoweld" Splice by
Exomet, Inc.
Conneaut, Ohio

EXOTHERMIC #2

"Thermit" Splice by
Thermex Metallurgical, Inc.
Lakehurst, N. J.

EPOXY #1

"Colma-Dur Gel" Bonding Compound by
Sika Chemical Corp.
Passaic, N. J.

EPOXY #2

General Purpose Adhesive,
Formulation 991-67
See body of report page 16

EPOXY #3

Thermoset 110 Epoxy Adhesive by
Thermoset Plastics, Inc.
5101 East 65th Street
Indianapolis 20, Indiana

EXPANDED GROUT prepared with
Embeco No. 5 by
The Master Builders Co.
Cleveland, Ohio.

Note:

It should be understood that the information contained in this report is the result of laboratory tests supervised by the author and does <u>not</u> in any way constitute an endorsement or approval of any trade name product nor a sanction against any trade name product by either the sponsors of the project, the University of Oklahoma Research Institute, or the author.

CHAPTER II. FABRICATION AND TESTING OF TENSILE SPLICES

General

The fabrication of the connections required a device to hold the two bars in rigid alignment during the splicing operation and a jig was constructed to fill this need (see Figure 2). This jig may be used to make either vertical or horizontal connections depending upon its position. The sleeve-with-metal-filler connections were made with the axis of the reinforcing bar in a vertical position with the jig set on its end, but all other connections were made with axis of the reinforcing bar horizontal. In the case of the sleeve-with-metal-filler splices either horizontal or vertical can be made. Although this investigation did not verify it, the supplier of this splice states that the splicing position does not effect the structural properties of the splice.

The jig was relatively light and portable which was convenient because the exothermic type connections had to be made out of doors. They spewed bits of molten metal and produced a great deal of smoke from the mold during the reaction which could be a definite disadvantage on the construction site. In contrast, the sleeve-with-metal-filler type connection generated a relatively small amount of heat, negligible smoke and the fire hazard from the spewing bits of reactant material was comparatively minimal.

Control Bars

Two control bars were tested of each size and yield point steel used in this investigation (see Table 1). The control bars consisted of 24" lengths cut from stock of the same heat used to make the other connections and were tested exactly the same way.

Connections

1. Butt Weld

Two butt welded connections were prepared and tested for each bar size and steel grade. In ordering the samples in this series of connections, each welded joint was specified to have sufficient strength so as to develop the full tensile strength of the bar per the American Welding Society specification D12.1, "Recommended Practices for Welding Reinforcing Steel, Metal Inserts, and Connections in Reinforced Concrete Construction." During the

NOMINAL YIELD STRENGTH (psi)	40,000				60,000				TOTAL SAMPLES TESTED
ASTM DESIGNATION	A-15	A-15	A-15	A-408	A-432	A-432	A-432	A-432	
BAR SIZE	#9	#10	#11	#14S	#9	#10	#11	#14S	
CONTROL BARS	2	2	2	2	2	2	2	2	16
BUTT WELDED	2	2	2	2	2	2	2	2	16
SLEEVE WITH METAL FILLER	3	3	3	3	3	3	3	3	24
EXOTHERMIC #1	3	3	3	3	3	3	3	3	24
EXOTHERMIC #2	3	3	3	3	3	3	3	3	24
EPOXY #1	5								5
EPOXY #2	5								5
EPOXY #3	3								3
EXPANDED GROUT	5								5
TOTAL NUMBER OF SAMPLES TESTED									122

TABLE I. TYPES AND NUMBER OF SAMPLES TESTED IN PART I testing program, a value of 125% of the nominal yield strength of the bar was also considered as per 1963 ACI Code Section 805(d).

2. Exothermic #1

The exothermic #1 consisted essentially of a sand mold filled with reactant powder which, when ignited, reacted rather violently and changed to a molten state which flowed by gravity around and between the ends of the bars. In making the connections, the procedure recommended by the manufacturer was followed. A sealing paste was applied to the contact faces of the mold halves to prevent leakage. The bars were aligned with a 3/8 inch gap between their ends by a spacer and the mold positioned so that the metal flow channel was directly over this gap. The ends of the mold around the reinforcing rod were luted with sealing sand and two metal disks were placed in the space provided at the bottom of the mold. The reactant powder was then poured into the mold and ignited. The reaction reached an extremely high temperature (4600°F) which melted the metal disks thus allowing the molten metal to flow between and around the bars so as to weld them together.

After a few minutes the mold, which is itself destroyed, can be removed but a longer waiting period is recommended to permit the refractory portion of the mold to stress-relieve the weld area. Removal of flash and excess metal is not required but may be accomplished if space limitations demand it.

3. Exothermic #2

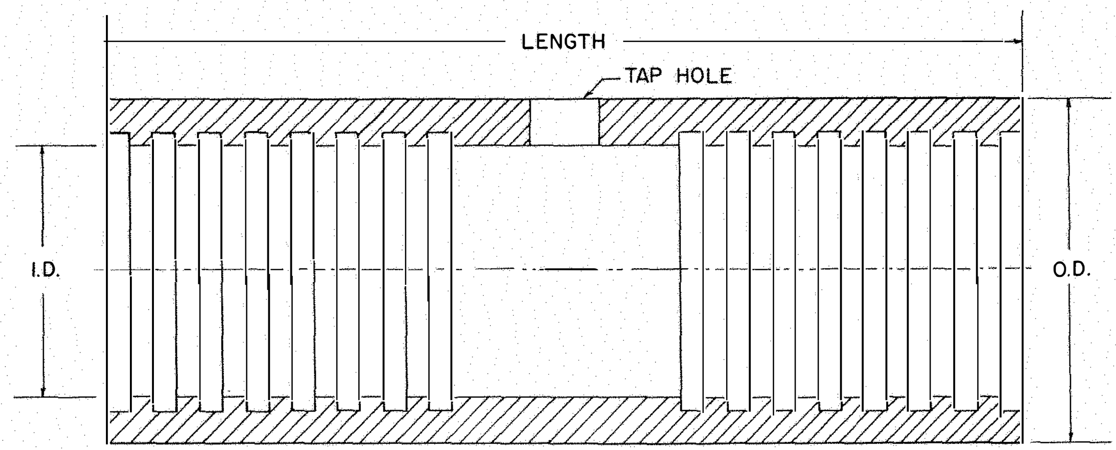
The exothermic #2 connection very closely resembles the exothermic #1 connection. Both used luted sand molds with a reactant powder, both required a 3/8" gap between bars, and both literally welded the bars together. There were some minor differences, however, mainly in the physical procedure of making the connection. The exothermic #2 used no sealing paste on the contact surfaces of each half m ld. A tight seal was obtained by clamping the half molds together with 2" C-clamps. Also, the molds fit rather loosely around the bars and small wooden wedges were placed at each end of the mold between the bar and the mold. This tended to stabilize the mold in the correct position and prevent rotation. After ignition of the reactant powder, the actual weld is formed in the same manner as the exothermic #1.

4. Metal-Filled Sleeve

This method of connecting reinforcing bars together consists of inserting the ends of both bars into a common sleeve and filling the sleeve with a metal filler material which mechanically locks the bars together. The procedure in this type of connection was also one that was recommended by the manufacturer. The ends of the bars must be clean, dry and free of rust, grease, dirt, etc. However, tightly adhering mill scale need not be removed. For vertical connections, as were used in this study, the bottom alignment fitting is positioned on the lower bar so that the gap between the bars will be at the center of the sleeve. Asbestos wicking or packing is placed around the bar at the top of the bottom alignment fitting (which supports the sleeve) and also at the top of the sleeve. The asbestos wicking at the top of the sleeve is positioned against the sleeve by the top alignment fitting. The pouring basin is then attached tightly around the small opening in the center of the sleeve. The crucible is then placed on top of the pouring basin. A small steel disk is placed in the bottom of the crucible over the tap hole and the filler cartridge reactant powder poured into the crucible. A small amount of starting powder is then placed on top of the reactant powder and ignited with a flint. Upon completion of the reaction the disk is melted, and the molten alloy metal flows into the sleeve through the small opening in the center of the sleeve. It cools, thus locking or keying the sleeve (through the internal grooves) to the reinforcing bars (through the bar deformations). It should be emphasized that this is a mechanical connection, not a weld. The sleeves used were proportioned in order to develop at least the ultimate tensile strength of each bar size and steel grade. Table 2 summarizes the geometric properties of the splice sleeve.

5. Sleeves with epoxy #1, #2 and #3

All epoxy connections used identical #9 bar sleeves as supplied by the manufacturer for the sleeve-with-metal-filler type connections for bars meeting ASTM-A15 specifications. The epoxy served as the locking material in the sleeve and connected the bars together much as the sleeve-with-metal-filler type connection. Although none of the epoxies were manufactured for this particular purpose, it was desired to investigate their



SLEEVE CROSS-SECTION

SPLICE SLEEVE						TENSILE STRENGTH OF SPLICE BASED ON AREA OF REBAR
BAR SIZE	NOMINAL YIELD STRENGTH OF REBAR (k.s.i.)	LENGTH (in)	O.D. (in)	I.D. (in)	AREA OF SLEEVE AT TAP HOLE (in^2)	$\frac{\text{AREA SLEEVE}}{\text{AREA REBAR}} \times 80{,}000$ p.s.i.
14 S	60 40	7 7	2-3/4 2-5/8	2 2	2.564 2.075	91,200 73,800
11	60 40	6 6	2-3/8 2-1/4	1-3/4 1-3/4	1.869 1.446	95,800 74,200
10	60 40	5 5	2-1/4 2-1/8	1-5/8 1-5/8	1.746 1.348	110,000 84,900
9	40 or 60	5	2	1-1/2	1.250	100,000

TABLE 2. PROPERTIES OF SLEEVES USED

possible value in this area. Because of the limited amount of epoxy #3 available, only three samples of this type were made, but five samples each were made using epoxy #1 and epoxy #2.

Epoxy #1 and epoxy #3 were commercial formulations, while epoxy #2 was formulation 991-67 (General Purpose Adhesive) as outlined in American Railway Engineering Association Bulletin #573. Formulation 991-67 consists of the following materials:

Epi-Reg 509	(Jones-Dabney Company)
Asbestos 7-TF-1	(Johns-Manville Asbestos Fibre Div.)
Alumina T-60	(Aluminum Company of America)
Epi-Cure 855	(Jones-Dabney Company)

When mixed, each epoxy had the consistency of a light weight grease.

The epoxies presented some special handling problems as the pot life averaged approximately 15 minutes and no special equipment was available to force it into the sleeves. In order to reduce the possibility of voids, each sleeve was filled with epoxy and then one bar was inserted, which forced out some excess epoxy. Then the second bar was inserted which forced out an additional excess quantity of epoxy. It was felt that a satisfactory procedure for filling the sleeves could be developed providing the preliminary results of these limited numbers of samples warrant it. The connections were tested one week from the date of fabrication after being cured at room temperature and humidity, and the stress-strain diagrams plotted where sufficient data resulted.

6. Sleeve with Expanded Grout

The sleeve-filled-with-grout connections also used the #9 bar sleeves supplied by the sleeve-with-the-metal-filler manufacturer. The bars were of ASTM-A15 material. The grout consisted of one part commercial expanding concrete additive and one part of type I portland cement. Enough water was added to render the mixture suitable for use in the sleeves. The method of fabrication was similar to the epoxy connections, i.e., one bar at a time was inserted into the grout-filled sleeve. Five samples were made and were tested at 28 days after curing in a moist chamber for 7 days and 21 days in air.

Testing Procedure

All tests conducted during this investigation were static tensile tests using a standard 8" gage length with the connection approximately centered within this length.

All bar cross sectional area calculations were based upon the measured diameter of the bar at the base of the deformations. The first #9 and #10 control bars were tested using a two point, one gage extensometer but this was to be unsuitable for the larger diameter bars or the sleeve type connections due to space limitations of the yoke. Consequently, a special type extensometer was designed and constructed. This consisted of two rings, 8" apart (c/c of bolts), each fastened rigidly to the test specimen by four radial bolts (see Figure 2). As two .0001" Ames dials were attached between the two rings and diametrically opposed to each other, the average total strain recorded would be that of the center line of the test specimen.

The loading rate was approximately 2000 lbs. per minute for all tests. Due to their dependability against sudden failure, the sleeve-with-metal-filler type connections were tested to the nominal yield point before the gages were removed. After damaging several Ames dials due to an unexpected early failure of an exothermic type connection, all subsequent test specimens were preloaded to approximately two-thirds of their nominal yield strength before the gages were set in place. When the connection had sufficient strength to elongate the bar an appreciable amount after removal of the gages, large dividers and a .01" scale were used to obtain strain measurements until failure. All tests were conducted on a 200,000 lb Baldwin Universal testing machine which was calibrated before the investigation began.

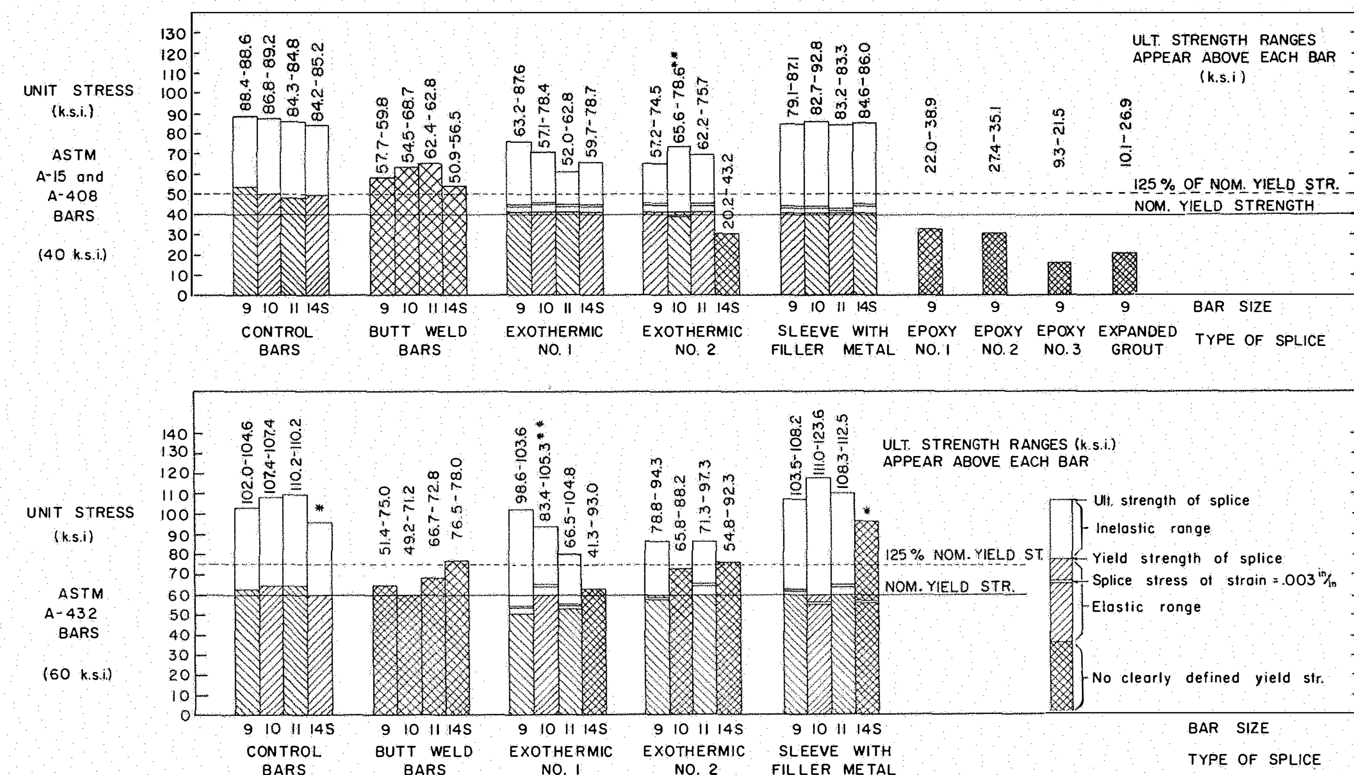
CHAPTER III. DISCUSSION OF TEST RESULTS

Control Bar

As expected, the control bars established the upper limit of values of load and stress. A summary of all test results is shown on Figure 1 of this report. Also note the stress-strain curves for each sample shown in Appendix Figures A-1 through A-58. Each sample is compared with a bar without a splice (labeled control bar or theoretical bar).

Butt-Welded Splice

The average values of the maximum loads indicate that the butt welded connections exceeded the 125% of nominal yield strength criteria for 40 ksi nominal yield steel but failed to do so for the 60 ksi yield steel. With the 40 ksi yield steel, Figure 1 indicates that the welds were satisfactory but tends to mask some rather erratic test results since an average of two samples is shown. The individual stress-strain curves in the appendix must be taken into account for an accurate analysis. All samples failed in the weld area which, in nearly all cases, showed porosity and a lack of full penetration.



* 1. Bars did not break as they exceeded the 200 kip limit of the testing machine.
2. Each control bar column and each butt weld column is the average of two test samples.
3. Each exothermic column and each sleeve with filler metal column is the average of three test samples.
4. Each epoxy column (nos. 1 and 2) is the average of five test samples using the same sleeve as used with the metal filled sleeve.
5. Each epoxy column (no. 3) is the average of three test samples using the same sleeve as used with the metal filled sleeve.
6. For No. 9 bars butt welded, insufficient data was available to determine yield strength or unit stress corresponding to .003 in/in of strain.
7. Epoxy no. 3 samples all failed during the preload test of approximately one-half the yield strength.
8. Expanded grout column is the average of five test samples using the same sleeve as used with the metal filled sleeve.
** 9. One sample separated when the mold was removed.

FIGURE 1. SUMMARY OF SPLICE TEST RESULTS.

Exothermic #1 Splice

All exothermic #1 connections proved satisfactory with the exception of the #14S bars of 60 ksi nominal yield strength. An average of these three samples gives an ultimate strength of just over the nominal yield strength.

All but two samples failed in the connection itself. An examination of the cross section after failure revealed a large variation in appearance ranging from uniformly solid to a honeycombing affect with relatively large voids. Oddly enough, no correlation could be established between the presence or absence of the voids and the ultimate strength. No explanation is offered here. The two samples that did not fail in the connection itself were the #9 bar connections and the bar failed in both cases. One bar was of 40 ksi nominal yield strength and the other was of 60 ksi nominal yield strength.

It should also be noted that one of the 24 exothermic #1 connections completely failed to join the bar together. When the mold was removed, the bars simply fell apart. The reason for this was not determined. The tips of both bar ends were covered with reactant metal which seemed to indicate that they were approximately in the correct position with respect to the metal flow channel.

Exothermic #2 Splice

In general, the exothermic #2 connections were not as satisfactory as the exothermic #1 connections. In the 40 ksi steel grade group, the No. 14S bars failed prematurely. The reason for this is difficult to determine. The break, which always occurred in the general weld area, appeared solid and free of voids but had a very even, fine-grained appearance somewhat different from the control bar breaks. Some failures were located slightly outside of the original gap between bars, indicating that the tip of one bar had separated. However, the average ultimate stresses of the other bars in this group were all above the 125% yield strength level.

Like the exothermic #1, one of the 24 exothermic #2 connections was a total failure and fell apart when the mold was removed. However, the reason seemed to be a leak in the mold which allowed considerable molten metal to be lost.

Sleeve-with-metal-filler Splice

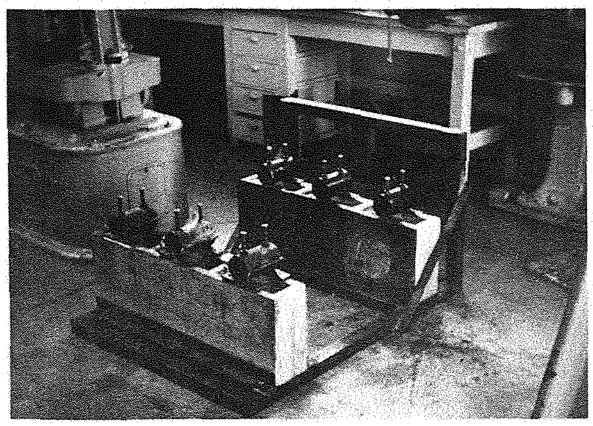
For ultimate strength and reliability, this connection equalled the control bars. There were no premature failures. Of the 24 samples tested, only two sleeves failed and then at very high stresses. In all other samples the bars themselves failed and did so at a stress comparable to that of the control bars. Like the No. 14S control bars of 60 ksi nominal yield strength steel, the comparable connections did not break but reached the 200,000 lb limit of the testing machine (corresponds to a stress of approximately 89 ksi).

Strain may prove detrimental when joining the higher strength bars as the 0.003 inch per inch strain was reached before the nominal yield strength in two of the four sizes of bars tested in the 60 ksi group. All bars of 40 ksi material passed the nominal yield strength before reaching this strain.

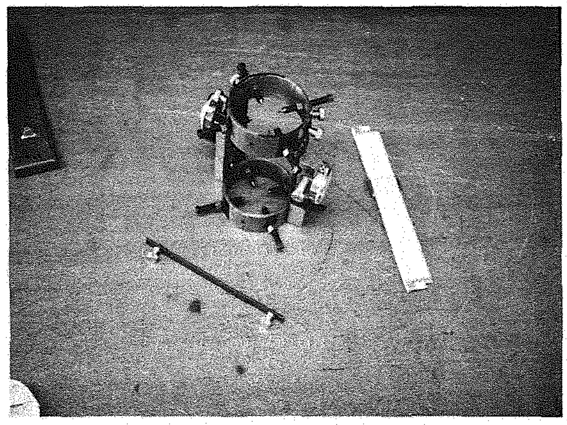
Sleeves filled with Epoxy #1, #2, #3 and Expanded Grout.

This was a splinter project of this study. As epoxies are fairly new and apparently quite versatile, it was decided to examine the feasibility of using them as filler material for sleeve-type tensile connections. For the same reason, a sleeve filled with commercial expanded grout was tested. All tests failed at such disappointingly low loads that this area was not considered further. Each sample failed in shear between the inside sleeve surface and the surface of the reinforcing bar.

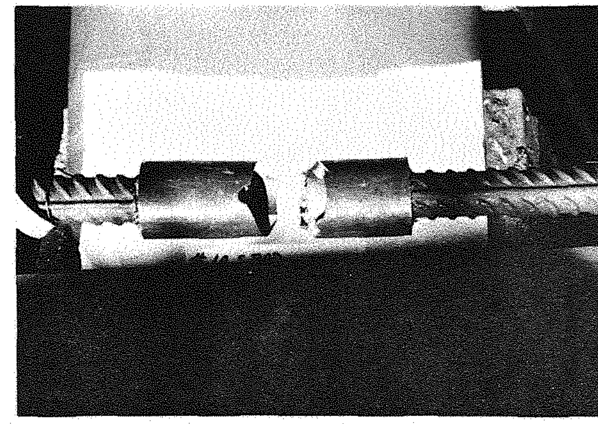
Photographs of the test equipment and typical test samples are shown on the following page in Figure 2.



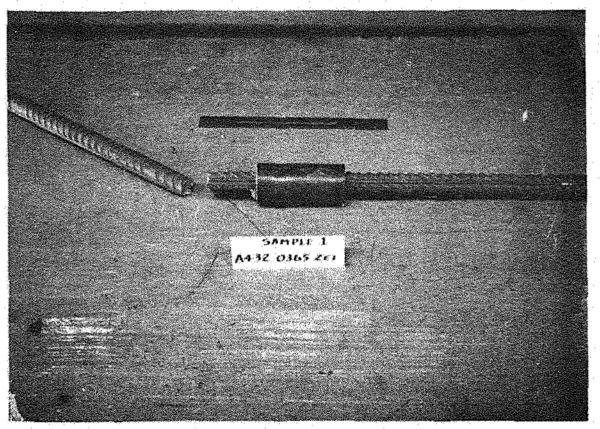
a. Alignment jig.



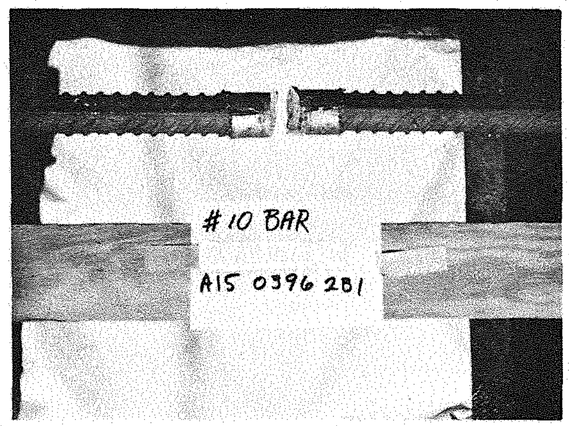
b. Tensile Extensometer.



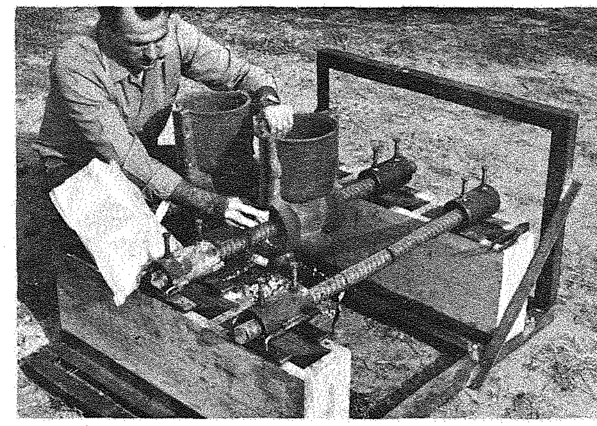
Typical sleeve failure in sleeve with metal filler type connection.



d. Typical bar failure in sleeve with metal filler type connection.



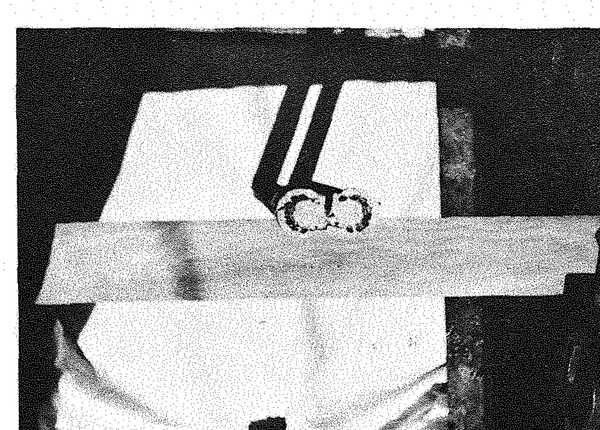
e. Typical Exothermic No. 2 failure.



Typical exothermic splicing operation.

g. Exothermic No. 1 connections being formed.

h. Surplus slag metal on Exothermic No. 1 connection before removal.



i. Typical cross-section of Exothermic No. 1 connection after failure.

FIGURE 2. Test photographs (Part I)

CHAPTER IV. PRELIMINARY CONCLUSIONS

Butt-Welded Splices

Because of the wide variation in the test results in this study as well as the cost involved, this type of splice was not completely satisfactory. This is especially true in the case of the ASTM-A432 bars. Under ideal conditions ASTM-A15 and ASTM-A408 butt-welded splices may be acceptable.

Sleeve-with-metal-filler Splices

Based on the results of this study, the sleeve-with-metal-filler type connection will for all practical purposes equal the ultimate strength of both 40 ksi and 60 ksi nominal yield strength bars. Their consistently high quality was remarkable, and, if the recommended procedure is followed, it appears that this splice can be used with complete confidence. However, the allowable strain may be a limiting factor when the higher strength ASTM-A432 bars are used as the metal filler allows more strain than the reinforcing bars. Thus, some of the advantage of high strength bars may be lost if local concrete cracking around the connection results. With ASTM-A15 and

ASTM-A408 bars the nominal yield strength of the steel was reached before the connection yielded 0.003 inches per inch and excessive strain does not appear to be a problem.

A very definite advantage of this type connection seems to be that its quality can be ascertained with accuracy by visual inspection, i.e., if the filler metal is visible at each end of the sleeve after removal of the asbestos packing, it can reasonably be assumed that the sleeve contains the proper amount of filler. In addition the connection is relatively quick and easy to make since only approximately five minutes is required before the equipment can be removed immediately for the next set up. As the sleeve and filler metal are the only items consumed, no troublesomecleanup is necessary. The fire hazard is minimal due to the relatively mild reaction in the crucible and the use of a splash guard.

Exothermic Splices

Based on the samples tested, these connections do not appear to possess the high reliability of the sleeve-with-metal-filler type connection but the exothermic did give reasonable average values of maximum stress. However, the averaging of samples tends to hide a wide variation between individual samples. For example, Exothermic #1 splice of 40 ksi nominal yield strength steel failed at stresses ranging from approximately 52,000 psi to 88,000 psi and from 41,000 psi to 105,000 psi for 60 ksi nominal yield strength steel. The Exothermic #2 splice of 40 ksi nominal yield strength steel failed at stresses ranging from approximately 20,000 psi to 79,000 psi and from 55,000 psi to 97,000 psi for 60 ksi nominal yield strength steel.

Both of the exothermic splices strained relatively very little up to the point of failure which from the standpoint of deflection is a real advantage.

A definite disadvantage of this type connection appears to be its immunity from an accurate visual inspection. Very little can be determined about its load carrying capabilities simply by looking at it. All samples made during this research looked identical, but they failed over a wide range of stresses.

More time is required to make the exothermic connections than the sleeve-with-metal-filler type. Both exothermic type connections are very similar and both require approximately ten minutes per connection but the clean-up time, i.e., removing the molds, could extend this time somewhat. In addition, the fire hazard in the immediate vicinity of the reaction could be a problem around wooden formwork.

Epoxy #1, #2, #3 and Expanded Grout Splices

On the basis of this study, these connections appear to be totally inadequate and should not be used for tensile splicing reinforcing bars unless the length of the splicing sleeve is significantly increased as compared to the sleeve used in the metal-filled sleeve splice.

SELECTED REFERENCES

1. Abrams, Duff A., "Tests of Bond Between Concrete and Steel, "University of Illinois Engineering Experiment Station Bulletin No. 71, December, 1913.

2. Chamberlin, S.J., "Spacing of Spliced Bars in Beams," Journal of the American Concrete Institute, Volume 29, No. 8, February, 1958, Proceedings Volume 54, pp. 689-697.

3. Chinn, James, Ferguson, Phil M., and Thompson, J.Neils, "Lapped Splices in Reinforced Concrete Beams," Journal of the American Concrete Institute, Volume 27, N. 2, October 1955, Proceedings Volume 52, pp.201-213.

4. Clark, Arthur P., "Bond of Concrete Reinforcing Bars," Journal of the American Concrete Institute, Volume 21, No. 3, November, 1949, pp. 161-183.

5. Eriksson, Owe, "The Sleeve Method of Splicing Reinforcing Bars," Ingenioren-International Edition, Volume 4, December, 1960.

6. Ferguson, Phil M., Turpin, Robert D., and Thompson, J. Neils, "Minimum Bar Spacing as a Function of Bond and Shear Strength," Journal of the American Concrete Institute,Volume 25, No. 10, June 1954, pp.869-887.

7. Kluge, Ralph W. and Tuma, Edward C., "Lapped Bar Splices in Concrete Beams," Journal of the American Concrete Institute, Volume 17, No.1, September,1945,pp.13-36.

8. Mylrea, T.D., "Bond and Anchorage," Journal of the American Concrete Institute, Volume 19, No. 7, March,1948, Proceedings Volume 44 pp. 521-527.

9. Walker, William T., "Laboratory Tests of Spaced and Tied Reinforcing Bars," Journal of the American Concrete Institute, Volume 22, No.5, Jan., 1951, Proceedings Volume 47, pp. 365-375.

10. Watstein, David, "Bond Stress in Concrete Pull-Out Specimens," Journal of the American Concrete Institute, Volume 13, No. 1, September, 1941, pp. 37-50.

PART II

FLEXURAL BEAM TESTS

CHAPTER V. OBJECTIVES AND SCOPE

Objectives

It was the primary objective of Part II to determine the effectiveness of the more promising types of splices as determined from Part I when the splices were imbedded in concrete and acting as a flexural member as compared to a member which contained no splice -- rather a continuous reinforcing bar. A secondary objective was to compare the deflection of a member and the cracking pattern which might develop in the concrete because of the nature of the splicing device itself as compared to a member without such a splice.

Scope

For the initial study, the flexural members which would be cast would be limited to members containing one, No. 11, ASTM-A15 bar when loaded as a simple beam and subjected to two symmetrical concentrated loads, thus producing a constant moment section under live load conditions. The member would be subjected to a static load for relatively short time tests.

Initially it was decided that a total of six beams would be cast for this series of flexural tests. One beam would be a control sample, two beams would contain lap splices of different lengths, two more beams would contain a reinforcing bar spliced with a sleeve and filler metal, while another beam would consist of a reinforcing bar spliced with an exothermic No. 1 splice. Later, however, it was decided that two additional lap splice samples would be cast in which stirrups would be supplied in the lap region. Thus, a total of eight beams were included in this series of tests.

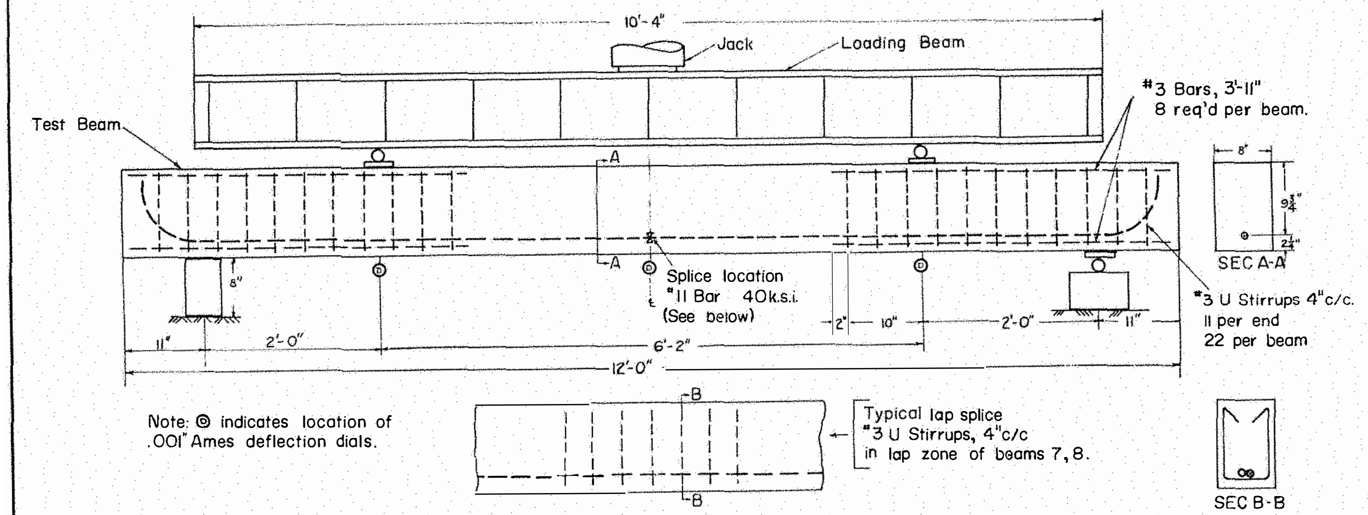
It should be pointed out that for this part of the study, the exothermic No. 1 device was selected rather than the exothermic No. 2 device because of its more consistent performance in Part I of this report. However, it is believed that very little difference would have resulted in the flexural tests planned regardless of which of the two exothermic devices had been selected - barring a premature failure of the splice itself.

CHAPTER VI. TESTING PROGRAM

Test Samples

The testing program was performed on the University of Oklahoma campus with the loading frame utilizing a hydraulic jack to apply the necessary load which was in turn reacted by the test frame.

Figure 3 outlines the details of reinforcement in each beam, as well as the type and location of splices which were utilized in the eight test samples. Table 3 summarizes the beam properties, including the details of the concrete mix used. The mix was designed to yield a 3000 psi concrete with Type III (High Early-Strength) cement being used for all test samples. Table 3 also outlines the compressive strength of the concrete used for each test sample as determined from the average of three 6"x12" concrete cylinder tests. The tensile strength of the concrete is also recorded based on the average of three 6"x12" split-cylinder tests. The reinforcing bars used for primary reinforcement in all beams were ASTM-A15, deformed bars of No. 11 size and were all from the same heat. Also from the same heat was the coupon sample that was used to experimentally determine the actual yield strength (49 ksi)



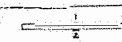
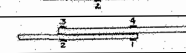
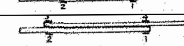
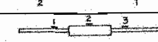
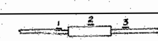
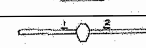
Beam No.	Type of connection	#3 U Stirrups in lap zone	Strain gage locations. All gages on longitudinal rib. (¼")
1	Control, no splice	—	
2	Lap 25D = 35"	—	
3	Lap 17D = 24"	—	
4	Sleeve, metal filler	—	
5	Sleeve, metal filler	—	
6	Exothermic #1	—	
7	Lap 25D = 35"	10 at 4"c/c	none
8	Lap 17 D = 24"	7 at 4"c/c	none

Figure 3. Reinforcement details, Flexural test samples

TABLE 3. BEAM AND CONCRETE PROPERTIES

BEAM NO.	ACTUAL WIDTH, inches	ACTUAL EFF. DEPTH, inches	CLEAR COVER, inches	f_c' [a] p.s.i.	STEEL YIELD STRENGTH, [b] k.s.i.	TENSILE STEEL USED	TYPE OF SPLICE	CONCRETE SLUMP, inches	CONCRETE FACTOR, [c] sacks/yard	MAX. SIZE AGGREG., inches	f_{sp}' [a] p.s.i.	$f_{sp}'/\sqrt{f_c'}$
1	8.0	9.75	1-1/2	2830	49.0	1-#11	None	5	5.0	1-1/4	377	7.1
2	8.2	9.75	1-1/2	3380	49.0	1-#11	25 D Lap[d]	5	5.0	1-1/4	452	7.8
3	8.2	9.85	1-9/16	2860	49.0	1-#11	17 D Lap[d]	5	5.0	1-1/4	381	7.1
4	8.0	9.85	1-1/2	3460	49.0	1-#11	Sleeve with filler metal	5-1/2	5.0	1-1/4	343	5.8
5	8.0	9.75	1-1/2	3200	49.0	1-#11	Sleeve with filler metal	5-1/2	5.0	1-1/4	355	6.3
6	8.0	9.85	1-1/2	3530	49.0	1-#11	Exothermic No. 1	5-1/2	5.0	1-1/4	366	6.2
7	8.0	9.85	1-1/2	2950	49.0	1-#11	25 D Lap[d] (stirrups)	4-1/2	5.0	1-1/4	300	5.5
8	8.0	9.85	1-1/2	2710	49.0	1-#11	17 D Lap[d] (stirrups)	4-1/2	5.0	1-1/4	324	6.2

[a] Average of three 6" x 12" cylinder tests.
f_c' = compressive strength of the concrete in the beam determined by compressive cylinder tests of the same age.
f_{sp}' = tensile strength of the concrete in the beam determined by split cylinder tests of the same age.
[b] Experimentally determined values of sample coupons from the same heat. (ASTM—A 15 Bars)
[c] High-Early Strength Cement (Type III) was used in all beams.
Fineness Modulus of the sand was 2.64. Aggregate was crushed limestone.
[d] Effective bonding lap reduced by approximately two bar diameters due to instrumentation attachments.

of the No. 11 bars. (See Figure A-65). The deformation pattern of all bars used in Part II were of the same pattern as used throughout Part I of the project.

It should be noted that the geometry of the test samples was such that by elastic theory, each beam was an over-reinforced or shallow beam. However, by ultimate strength theory, each beam was under-reinforced or a deep beam. This permitted the steel, tensile splices to be subjected to its ultimate load as represented by a yielding of the flexural steel and/or the splicing devise.

Beams 1, 2, and 3 were all poured as a single group, as were beams 4, 5, and 6. Likewise, beams 7 and 8 were poured as a third group. The forms were constructed so that the tension steel was in the bottom of forms on $1\frac{1}{2}$" bolsters. Each beam and its six companion cylinders were then cured for approximately four days under a plastic cover after being treated with a curing compound to prevent the loss of internal moisture. At the end of this period, each member and companion cylinders were stripped from their forms and allowed to cure under a plastic cover for at least five additional days before testing. For each test beam the companion cylinders were tested on the same day as the beam so as to insure their representing accurately the same concrete properties as existing in the member itself. After placing a member in the test frame, it was whitewashed with a lime and water preparation so as to add to the visibility of the cracks as they appeared during the testing operation.

Instrumentation

In all tests, .001" Ames deflection gages were mounted at the center of the span, as well as under each of the two concentrated loads.

In addition on beams 1 through 6, electric strain gages were placed on the reinforcing bars in the vicinity of each splice in the constant moment zone (Figure 3) so that the actual stress in the reinforcing bars could be measured at all times. In the case of beams 7 and 8, only deflection readings were recorded. All the SR-4 electric strain gages that were used in this project were mounted on the longitudinal ribs of the reinforcing bars and were 1/4" metal foil gages. Each gage was waterproofed with an epoxy waterproofing compound and protected with plastic tape around the bar at each gage location.

The location of electric strain gages for beams 1 through 6 are as follows (Also see Figure 3):

a) Beam 1: Two gages were placed directly at the center of the span on opposite sides of the reinforcing bars on the longitudinal ribs of the bar.

b) Beams 2 and 3: Four strain gages were used, two on each bar, so that one gage would be on the longitudinal rib at the termination of the bar and a second gage was mounted on the same bar at the point opposite the point of termination of the other bar.

c) Beams 4 and 5: Three strain gages were used in the longitudinal direction. One was mounted directly in the center of the splicing sleeve, while the other two gages were mounted on the longitudinal ribs of the reinforcing bars approximately one inch outside of the end of the sleeve.

d) Beam 6: Two strain gages were used. They were mounted in the longitudinal direction approximately an inch and one-half to two inches outside of the splice zone on the longitudinal rib of each reinforcing bar (approximately 6" apart).

Special attention should be given to beams 2 and 3. In each case the use of the plastic tape around the reinforcing bar at each strain gage location reduced the effective lap of each splice by 2 bar diameters or to approximately 23-D and 15-D for beams 2 and 3 respectively.

In order to duplicate the effective lap lengths accurately in beams 7 and 8, similar plastic tape at similar locations was added although no strain gages were installed. Thus the effective lap for beams 7 and 8 was also 23-D and 15-D respectively.

The reader will note that the 17 diameter lap is the minimum lap acceptable per 1961 AASHO Specifications, Section 1.7.5(c). The 25 diameter lap was also selected since it is the minimum lap used by the Oklahoma State Highway Department.

Procedure of Testing

During the testing of each beam, readings were taken from the deflection dials and the internal strain gages at various intervals within the elastic range. In addition at each interval loading, an effort was made to determine the cracks which had occurred since the last interval loading had been added. These cracks were marked with a black grease pencil to show the propagation of the cracks on the sides and the bottom of each member. This can be seen in Figures 6 and 7. Upon reaching approximately 75% of the expected ultimate capacity of the member, the deflection gages were removed while strain gages continued to be monitored as long as they were within their effective ranges. In each case the member was loaded to failure so that the ultimate load was determined. This process was repeated for each beam tested with the exception of beams 7 and 8 on which no electric strain gages were mounted.

CHAPTER VII. TEST RESULTS AND DISCUSSION

A summary of all results as well as the beam loading properties can be found in Table 4 of this report. In addition, two sets of deflection curves are included in Figures 4 and 5. These curves represent the deflections of each test beam both at the center line of the span as well as directly under the point of each concentrated load. Further amplification can be seen in the photographs of each test sample as represented in Figures 6 and 7. Comments for each test sample are outlined below:

<u>Test Beam 1</u>.

The results of this sample agreed closely with what might be anticipated since it developed 105% of the theoretical ultimate moment. It also checked the theoretical deflection calculations rather closely as can be seen from Table 4. The failure of beam 1 was through yielding of the flexural steel. At failure, the member continued to deflect, but refused to carry additional moment.

<u>Test Beam 2</u>.

Test beam 2 collapsed at the midspan at only 49% of the calculated ultimate moment. It had a 19%

increase in deflection as compared to the control sample, beam 1. This ratio of deflections is based on the live load working moment. Failure was through bond in the lap joint.

Test Beam 3.

Test beam 3 was a lap joint similar to beam 2 except that its reinforcement lap was only seventeen bar diameters in length. It developed only approximately 34% of the theoretical ultimate capacity of the member assuming that no joint failure would have taken place. Its deflection was 26% greater than the deflection was at the corresponding load for beam 1, the control sample. Failure was initiated by bond failure in the lap joint.

Test Beams 4 and 5.

Failure in both of these samples was by yielding or deformation in the flexural steel followed by crushing of the compression concrete near midspan. Although one of the gages on the reinforcing bar in beam 5 indicated slippage, other evidence such as deflection fails to substantiate any slippage of the bar within the sleeve. Therefore, one might conclude that the slippage occurred between the gage and the bar itself. Each of the samples had a slight longitudinal split in the bottom of the beam directly under the splicing sleeve at failure. However, this split did not occur until nearly the ultimate load of the member had been reached and did not appear to be a serious split from the standpoint of reducing the ultimate capacity of the member or perpetuating additional cracks. The width of the cracks in both samples, while more numerous than in the case of beam 1, did not appear to be objectionable in size or number, although the deflection for beams 4 and 5 was 53-61% greater than the deflection of beam 1 at the same live load working moment. The ultimate load of each beam was 75-76% of the theoretical ultimate capacity of the beam. The crack travel on each span might suggest that the compressive concrete failure was produced by the continuously reduced compressive area resulting as the internal moment arm increased with the increased deformation of the tensile steel.

Test Beam 6.

The splice of beam 6 was exothermic splice No. 1 and developed 80% of its ultimate theoretical capacity. The measured center line deflection at the working live load moment was approximately 26% greater than for the control sample, beam 1, at the same load.

Its crack pattern was very similar to the pattern developed by beam 1.

Test Beam 7.

Test beam 7 was spliced with a twenty-five diameter lap splice with No. 3 U stirrups placed at four inches center-to-center throughout the lap zone. This member developed approximately 79% of its theoretical ultimate capacity which was considerably greater than the 49% developed by beam 2 which was identical but for the stirrups in the lap zone. Therefore the approximate 60% increase in capacity can be contributed to the placing of stirrups in the splice zone. The deflection was similar to that of beam 2 in that it exhibited a 23% increase in deflection over beam 1 when a similar moment was applied to the member. No electric strain gages were located on this sample. Its failure, unlike beam 2, where a bond failure occurred, failed through yielding or deformation of the flexural steel followed by a crushing of the compression concrete near midspan.

Test Beam 8.

The reinforcement in beam 8 was spliced with a seventeen diameter lap splice in addition to No. 3 U stirrups placed throughout the lap zone and spaced at four inches center-to-center. This member developed approximately 55% of its theoretical ultimate capacity which is once again approximately 60% greater than the ultimate strength developed by beam 3 which did not contain any stirrups in the lap zone. Its deflection, however, was 44% greater than that of the control sample carrying the same moment. This compares to 26% greater for beam 3 without stirrups in the lap zone. The additional deflection exhibited by this member as compared to beam 3 is not explained. The failure of beam 8 was characterized by a bond failure in the lap joint just as was the case of beam 3 which did not contain any stirrups in the lap zone. No electric strain gages were located on this beam.

For all eight members the theoretical steel stress, f_s, with a live load working moment applied was approximately 13,000 psi. The measured stresses for beams 1 through 6, which contained gages on the reinforcement bars, were in most cases greater than this theoretical stress. This may be partially explained due to questionable magnitude of depth of the cracked section as well as to shrinkage and creep stresses which elastic theory does not immediately account for, especially at working loads.

Although the cracks, as can be seen in Figures 6 and 7, were marked at periodic intervals during the testing, no effort was made to measure the width of the cracks.

TABLE 4. BEAM LOADING PROPERTIES AND SUMMARY OF RESULTS

BEAM NO.	TYPE OF SPLICE	CALC. WORKING MOMENT,[1] kip feet	CALC. ULT. MOMENT,[2] kip feet	MEASURED ULT. MOMENT, kip feet	MEASURED ULT. MOMENT / CALC. ULT. MOMENT	CALC. LL f_s at WORKING MOMENT, k.s.i.	AVERAGE MEASURED LL f_s at WORKING MOMENT, k.s.i.	CALC. ℄ DEFLECTION[3] at LL WORKING MOMENT, inches	MEASURED ℄ DEFLECTION at LL WORKING MOMENT, inches	RATIO OF ℄ DEFLECTIONS[4]	TYPE OF FAILURE
1	None	15.6	49.4	51.8	1.05	13.0	15.7	.174	.172	1.00	Yielding of flexural steel
2	25-D Lap[6]	15.6	51.8	25.2	.49	13.0	18.8	.174	.205	1.19	Bond failure in lap joint
3	17-D Lap[6]	15.6	49.9	17.0	.34	13.0	30.7	.174	.217	1.26	Bond failure in lap joint
4	Sleeve with filler metal	15.6	51.8	39.0	.75	13.0	20.2	.174	.263	1.53	Beams 4, 5, 6, and 7 Yielding of flexural steel followed by compressive concrete crushing.
5	Sleeve with filler metal	15.6	50.8	38.5	.76	13.0	4.1[5]	.174	.277	1.61	
6	Exothermic No. 1	15.6	51.8	41.5	.80	13.0	16.2	.174	.216	1.26	
7	25-D Lap with stirrups[6,7]	15.6	50.6	40.0	.79	13.0	Not measured	.174	.212	1.23	
8	17-D Lap with stirrups[6,7]	15.6	48.8	27.1	.55	13.0	Not measured	.174	.247	1.44	Bond failure in lap joint

[1] Approximate value based on 1961 AASHO Specifcations [dead load (DL) + live load (LL)]

[2] Based on $M_u = A_s f_y (d - \frac{a}{2})$ where $a = \frac{A_s f_y}{.85 f_c' b}$ Small variations due to change in f_c' and beam cross-sections.

[3] Based on the gross concrete area neglecting the steel and $E_s = nE_c$. (n=10), $E_s = 29 \times 10^6$ p.s.i.

[4] Ratio of the measured LL center-line (℄) deflections at working moment for each member as compared to beam No. 1 at its LL working moment.

[5] Either one end of bar in sleeve slipped temporarily or gage 3 itself slipped on the bar. (No other evidence suggests bar slippage.)

[6] Effective bonding lap approximately two bar diameters less due to instrumentation attachments.

[7] No. 3 U stirrups at 4" c/c within the lap zone.

All the data above except as noted include the effect of the applied load, the dead load of the beam, and the weight of the loading beam. The dead load moment for all beams was 1.8 kip-feet.

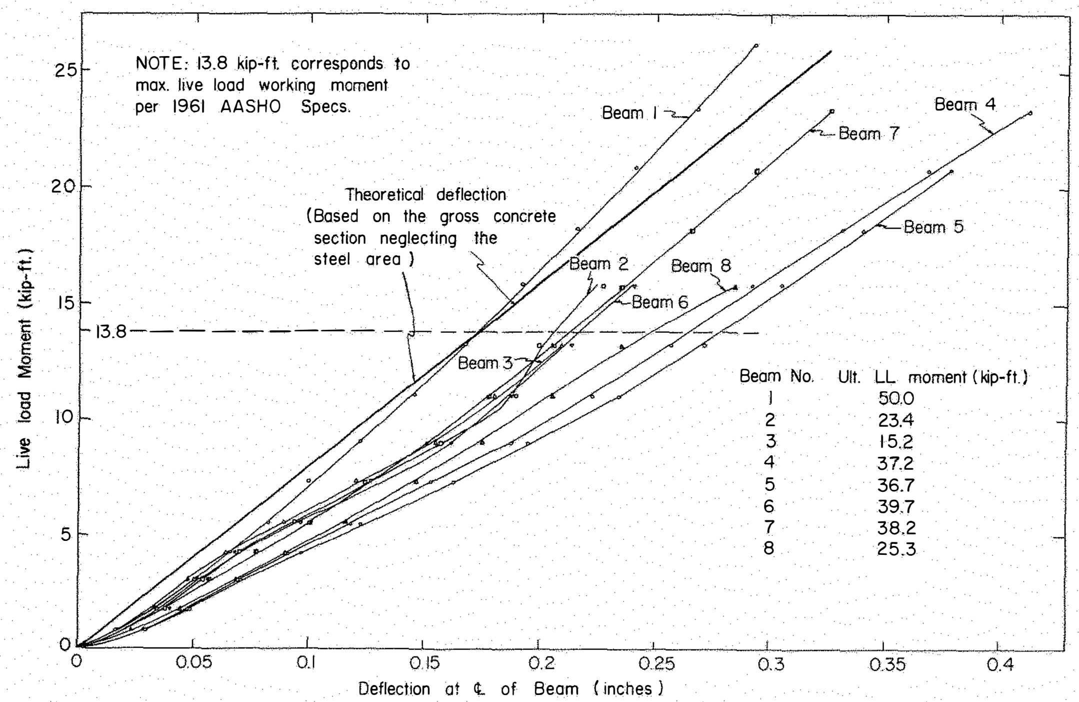


FIGURE 4. DEFLECTION AT MID-SPAN FOR EACH TEST SAMPLE.

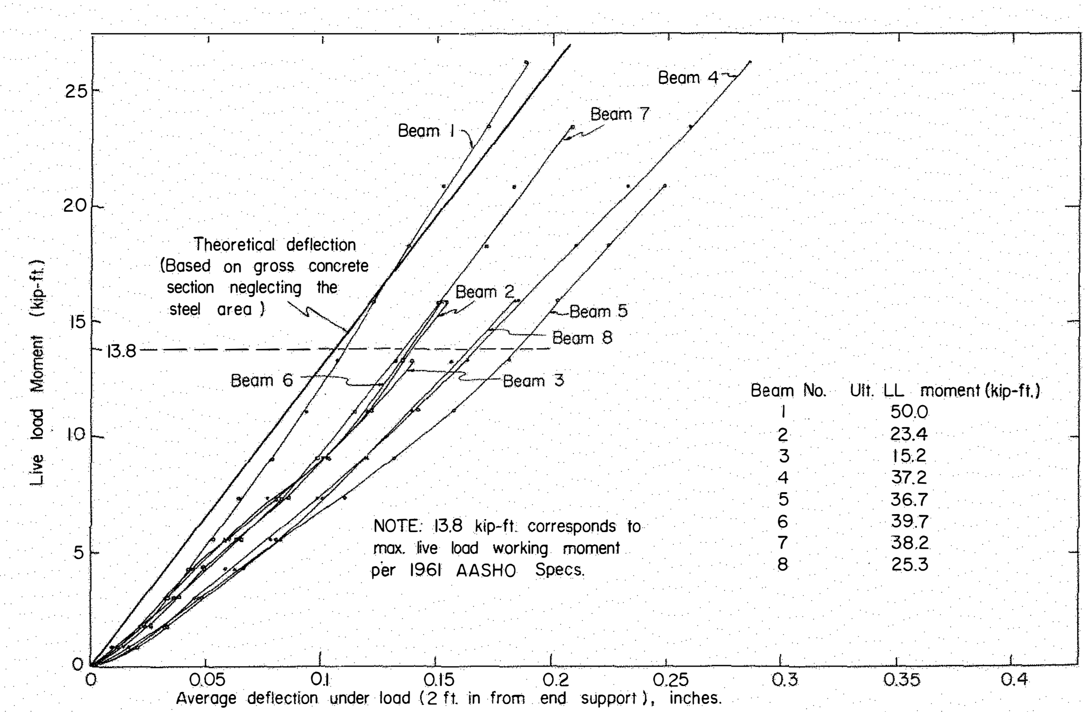
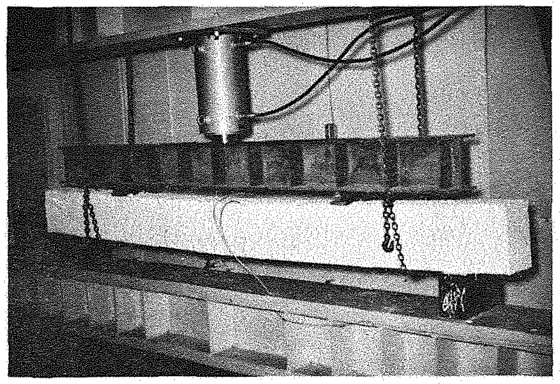
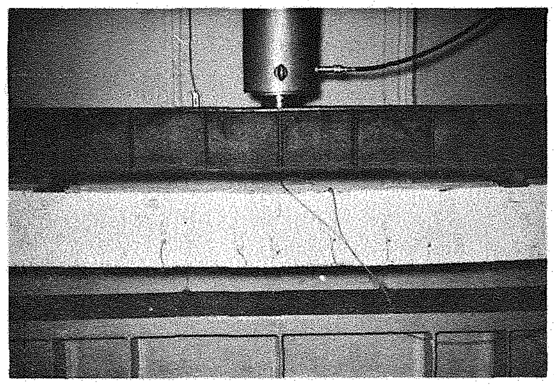


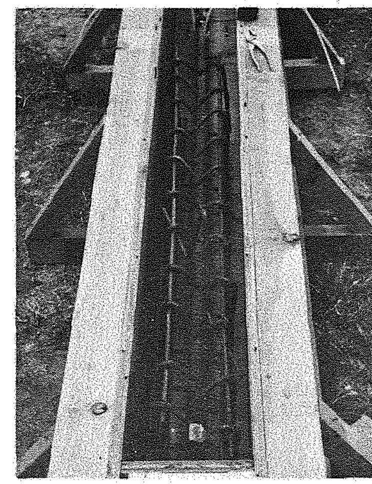
FIGURE 5. DEFLECTION UNDER THE LOADING POINTS FOR EACH TEST SAMPLE.



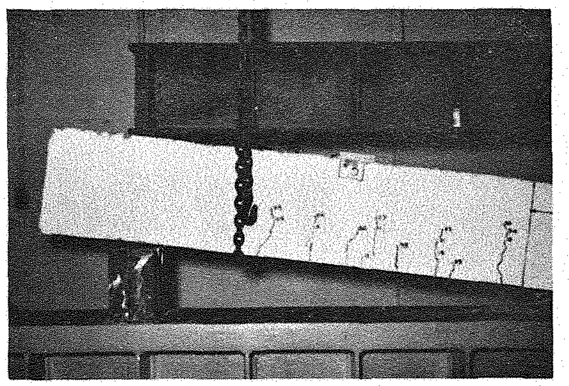
a. Test Beam 1 (control beam) in test frame.



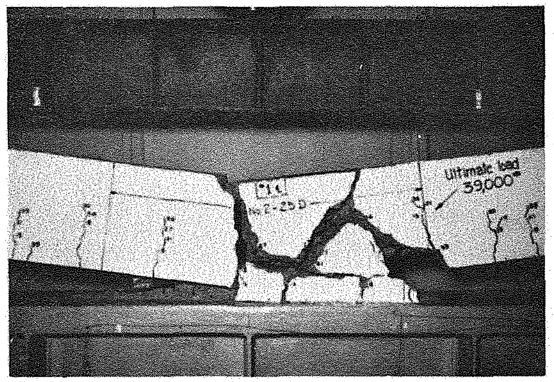
b. Crack pattern at failure in Beam 1.



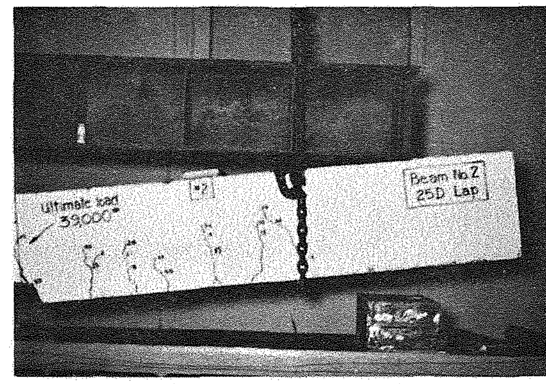
c. Beam 1 in the forms. Note strain gages.



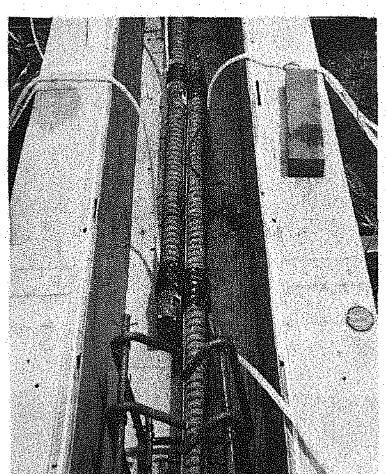
d. Beam 2 (left end) at failure.



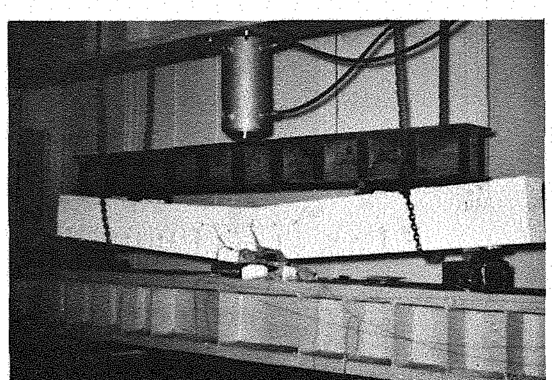
e. Lap failure in Beam 2.



f. Beam 2 (right end) at failure.



g. Beam 3 strain gage locations.

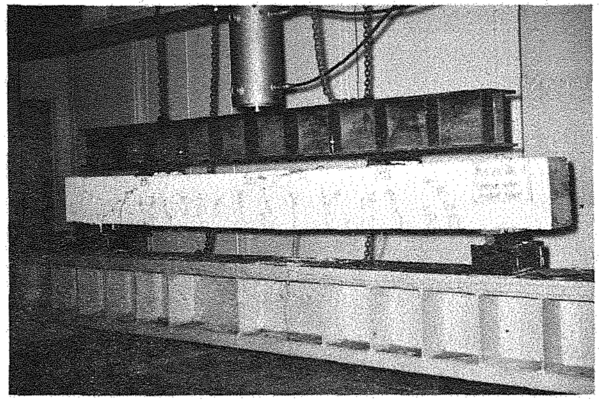


h. Beam 3 at failure.

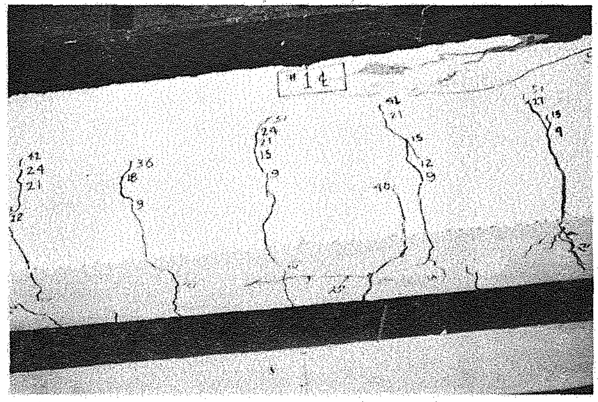
i. Lap failure in Beam 3

Note: Numbers by the cracks on each beam correspond to indicated jack load, not true load.

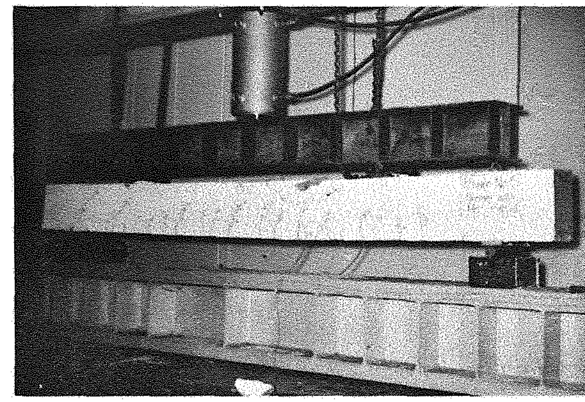
FIGURE 6. Test photographs of beams 1 through 3.



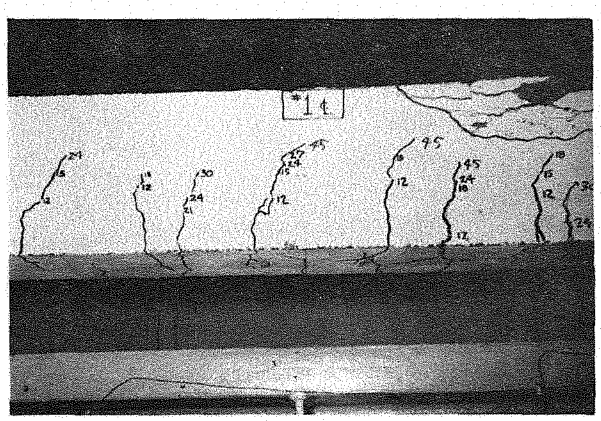
a. Beam 4 after failure.



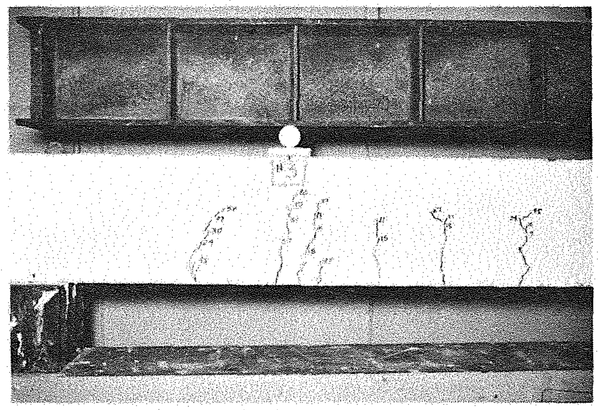
b. Cracks at the center on side and bottom of Beam 4 at failure.



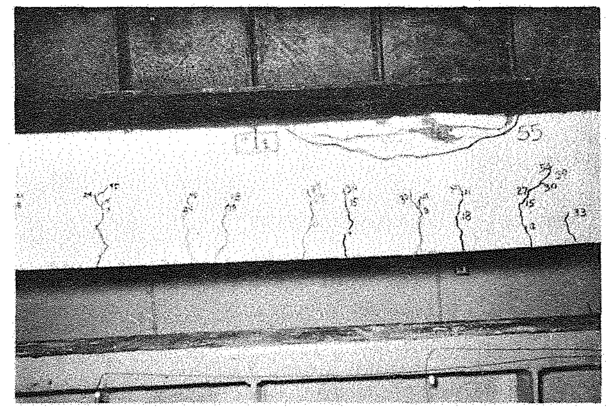
c. Beam 5 after failure.



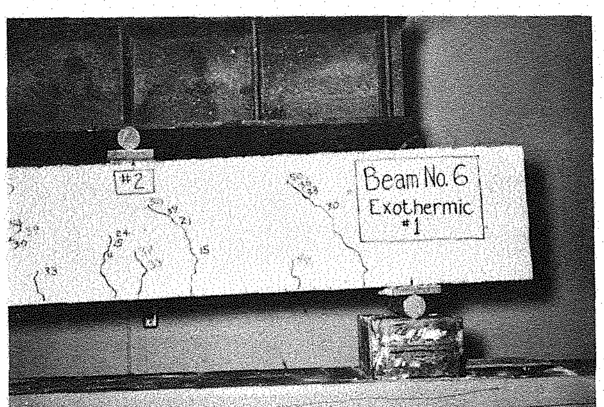
d. Cracks at the center on side and bottom of Beam 5 at failure.



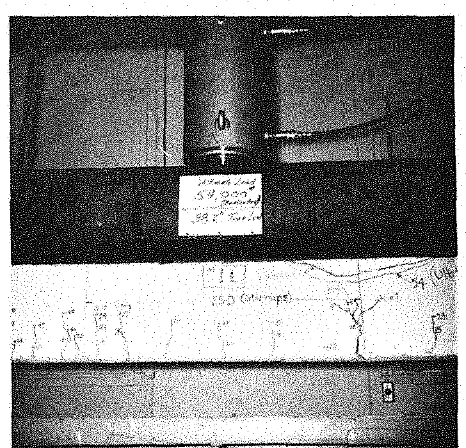
e. Left end of Beam 6 at failure.



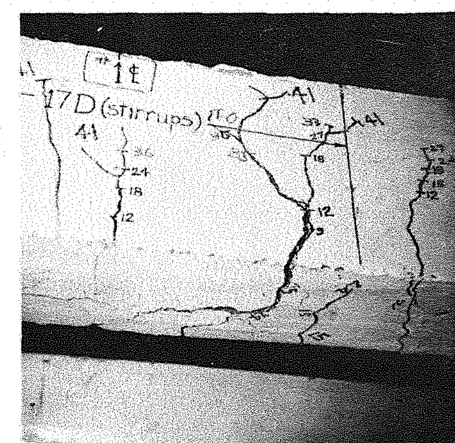
f. Center of Beam 6 at failure.



g. Right end of Beam 6 at failure.



h. Beam 7 after failure.



i. Bottom and side cracks in Beam 8 at failure.

Note: Numbers by the cracks on each beam correspond to indicated jack load, not true load.

FIGURE 7. Test photographs of beams 4 through 8.

CHAPTER VIII. CONCLUSIONS

Within the limitations of this study, the following statements appear to be warranted:

1. At this time there appears to be no equivalent substitute for a continuous bar in a member without sacrificing some ultimate capacity and stiffness.

2. At points where a splice must be used, greater deflection and reduced ultimate capacity should be recognized by the designer.

3. The sleeve-with-filler-metal, when subjected to a relative short-time static load, yields what appears to be a very satisfactory tensile splice, although at least 50% greater deflection can be anticipated with such a splice as compared to the absence of any splice at all, that is, a straight continuous bar. The ultimate capacity of the member so spliced may be only 75% of the member having continuous reinforcement.

4. The exothermic No. 1 splice would also appear to be a very satisfactory splice provided a reliable inspection procedure can be developed for the field so as to insure the reliability of any given splice without a physical test. In addition, more fabrication problems exist with the exothermic

devices than with the sleeve-with-filler-metal type. The deflection of the exothermic No. 1 device was comparable to the lap splice although at least 26% greater deflection resulted in members utilizing the exothermic No. 1 splice as compared to the control beam.

5. The 1961 Specifications of the American Association of State Highway Officials, Sections 1.7.5 (c) and 2.5.6, should be critically reviewed with the idea of possibly modifying it to substantially increase the lap distance required or reduce the allowable bond stress permitted to be used in the calculation of the required developmental length for such a splice. A lap distance greater than twenty-five bar diameters appears to be warranted.

6. The addition of stirrups in the lap zone at a spacing of not more than one-half the depth of the member appears to add approximately 60% to the strength of the lap joint without increasing the lap distance required.

7. The actual steel stress in the members studied exceeded the theoretical computed elastic stress by at least 20%.

8. Assuming that a lap splice is not feasible, a splice consisting of a sleeve-with-filler-metal appears to be the most suitable method for splicing tensile reinforcing bars (within the limitations of this study) when reliability,

strength, deflection, placement, and fabrication are all considered. At this time and on the basis of these tests, the exothermic devices appear to give a stiffer joint, although a less reliable joint, from the standpoint of inspection and consistency, than do the metal filled sleeve splices.

CHAPTER IX. RECOMMENDATIONS FOR FURTHER RESEARCH

1. The flexural tests in this study should be duplicated so as to include ASTM-A432 steel of No. 11 and 14S bar samples. This particular study is of considerable importance due to the additional deflection introduced by some of the mechanical connectors, which when coupled with the greater deflection inherent in using higher strength steels, may prove to be prohibitive for most design conditions.

2. Additional flexural tests should be undertaken in which there is not a constant moment section; that is, where shear is introduced as a variable. Such tests would clarify the effect of diagonal tension on splices as to whether or not the size of the sleeves in some splices precipitate additional diagonal tension cracks.

3. A limited number of samples might very well be studied using 5,000 psi concrete particularly where diagonal tension may be critical.

4. In most structures the splices under investigation would be subject to long-term, static, tensile loads. The effect of such loads should be investigated with particular

attention given to possible creep and the effect of such creep in the splicing devices on the member as a whole. The first part of any such investigation might consider only the bars (without being imbedded in concrete such as Part I of this study) when spliced in various ways and subject to long-term, static, tensile loads. The second part of such a study would involve the long-term static, loading of flexural members when the reinforcement in such members was spliced using various splicing devices. Once again a correlation with conventional lap splices would be indicated.

5. A fatigue study of reinforcing bars when spliced with various types of mechanical splices is also required. The prime importance here, as in the case above under long-term static tensile loads, would be to determine the amount of creep that might be expected in the splice itself when subjected to a fatigue loading.

6. Due to the relatively early failure of the lap splices in beams 2 and 3 of this study, it would certainly be well to consider an investigation of lap splices in general with particular emphasis on large diameter lap splicing such as No. 9, No. 10, and No. 11 bars with stirrups in the lap zone.

NOTATION

a = depth of the rectangular stress block and defined as $\frac{A_s f_y}{.85 f_c' b}$, inches.

A_s = cross-sectional area of the tensile steel, in^2.

b = width of the test beam, inches.

℄ = symbol for center-line

d = effective depth of the beam measured from the extreme compressive fiber to the centroid of the tensile steel, in.

E_c = modulus of elasticity of the concrete, psi.

E_s = modulus of elasticity of the reinforcing steel, taken as being 29,000,000 psi.

f_c' = unit compressive concrete stress based on ultimate strength 6"x12" cylinders tested on the same day as each beam test, psi.

f_s = unit tensile stress in the reinforcing steel, psi or ksi.

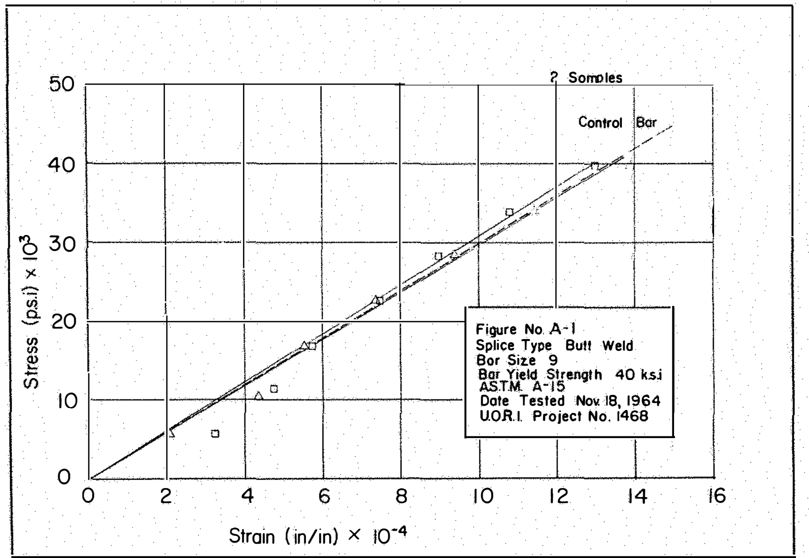
f_{sp}' = tensile unit strength of the concrete based on 6"x12" split cylinders tested on the same day as each beam test, psi.

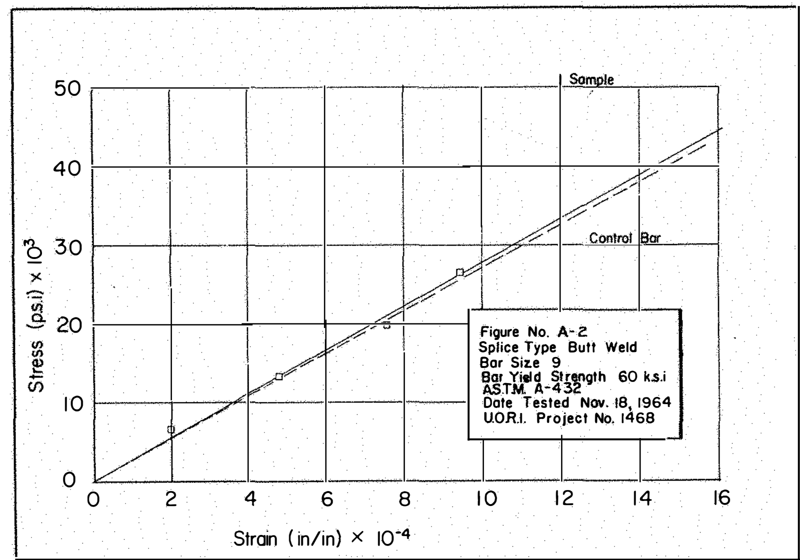
f_y = unit yield strength stress of the reinforcing steel, psi or ksi.

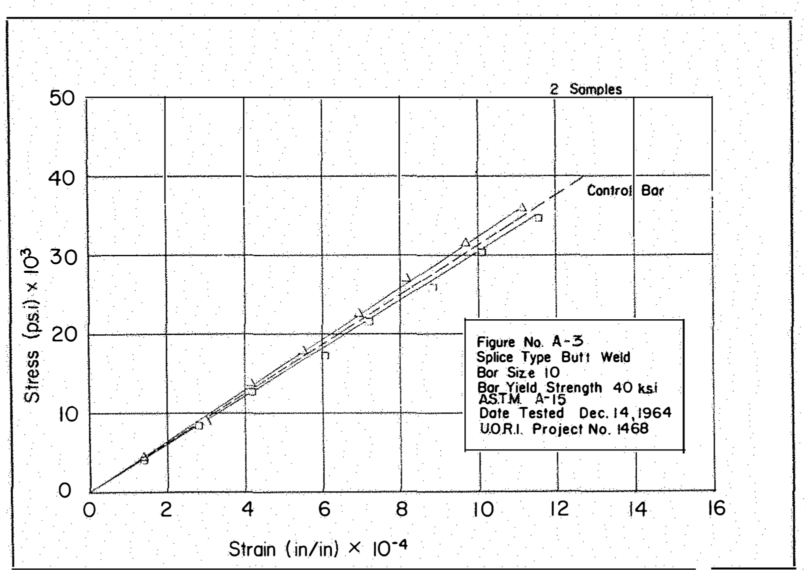
M_u = ultimate moment, ft.-kips or in-lbs.

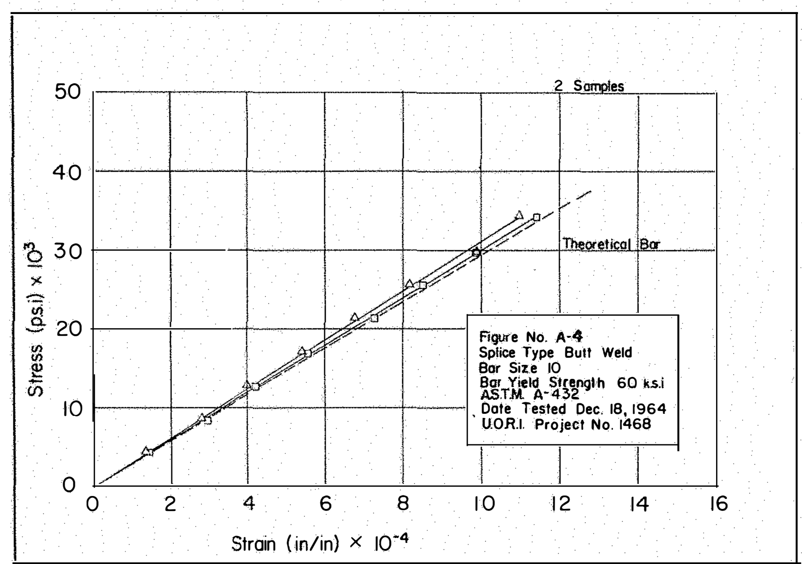
n = E_s/E_c

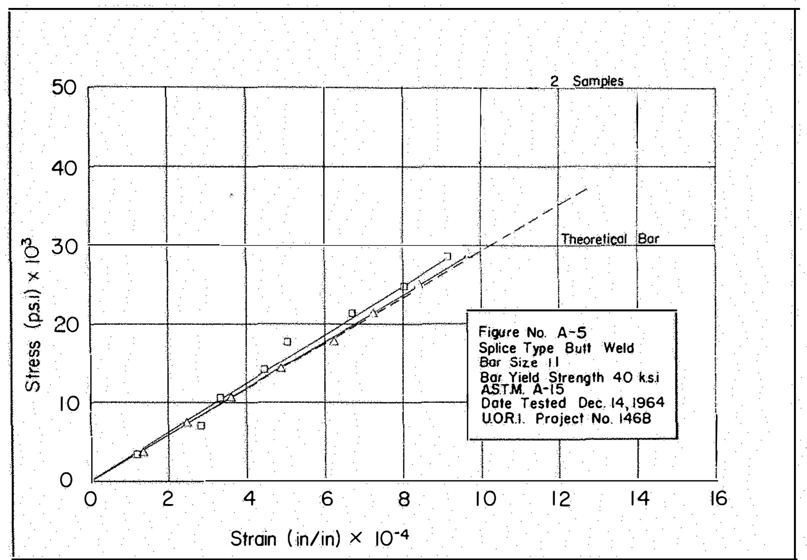
APPENDIX

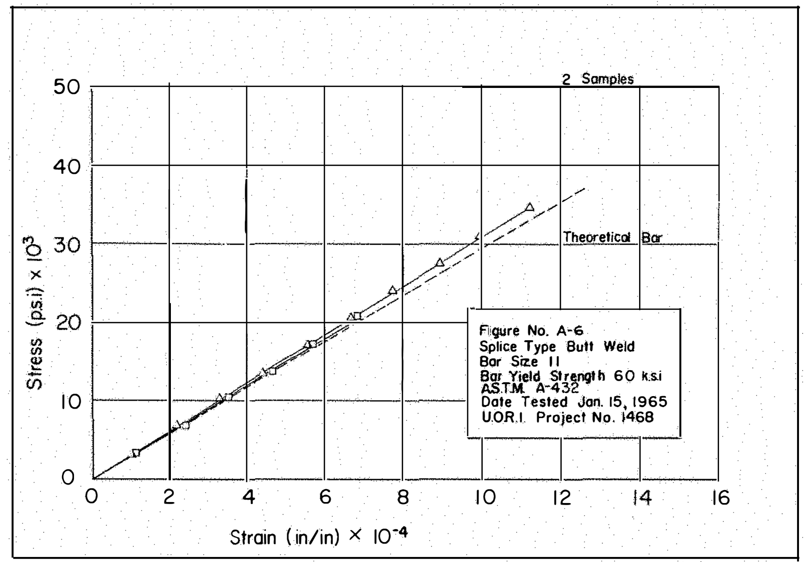

2 Samples
Control Bar
Stress (p.s.i) × 10^3
Strain (in/in) × 10^-4
Figure No. A-1
Splice Type Butt Weld
Bar Size 9
Bar Yield Strength 40 k.s.i
A.S.T.M. A-15
Date Tested Nov. 18, 1964
U.O.R.I. Project No. 1468

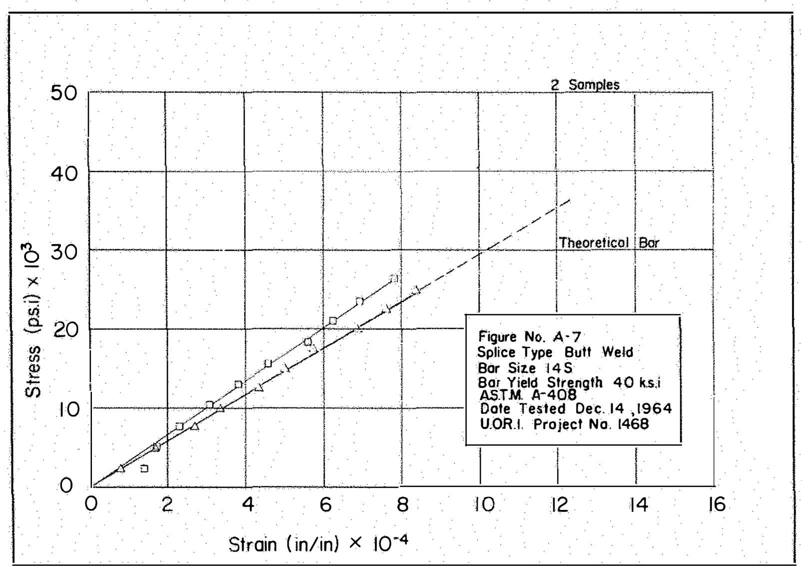

1 Sample
Control Bar
Stress (p.s.i) × 10^3
Strain (in/in) × 10^-4
Figure No. A-2
Splice Type Butt Weld
Bar Size 9
Bar Yield Strength 60 k.s.i
A.S.T.M. A-432
Date Tested Nov. 18, 1964
U.O.R.I. Project No. 1468

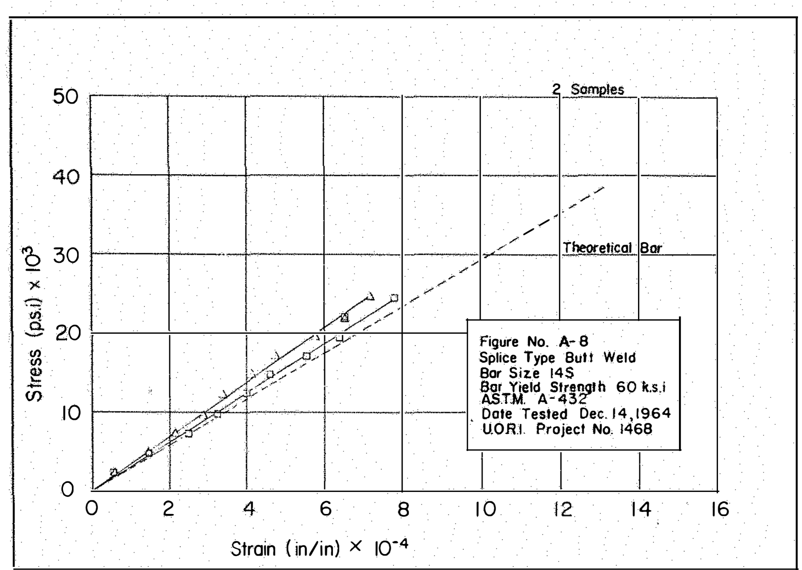

2 Samples
Control Bar
Stress (p.s.i) × 10^3
Strain (in/in) × 10^-4
Figure No. A-3
Splice Type Butt Weld
Bar Size 10
Bar Yield Strength 40 k.s.i
A.S.T.M. A-15
Date Tested Dec. 14, 1964
U.O.R.I. Project No. 1468

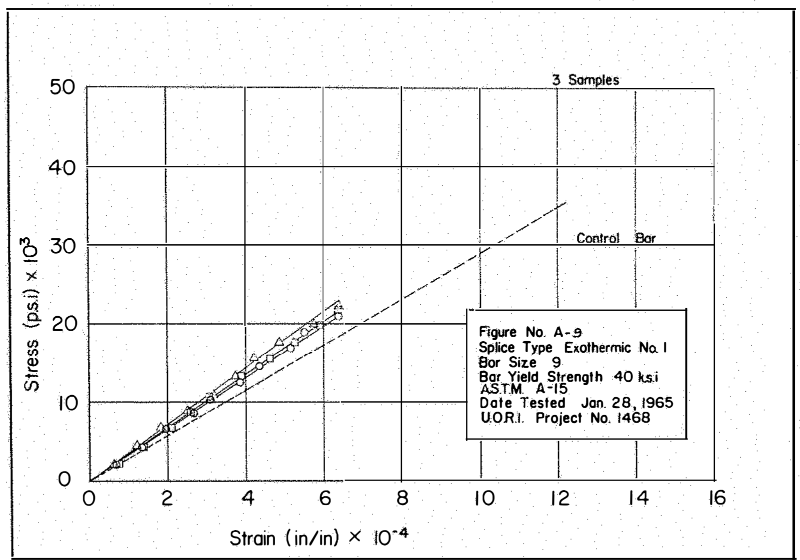

2 Samples
Theoretical Bar
Stress (p.s.i) × 10^3
Strain (in/in) × 10^-4
Figure No. A-4
Splice Type Butt Weld
Bar Size 10
Bar Yield Strength 60 k.s.i
A.S.T.M. A-432
Date Tested Dec. 18, 1964
U.O.R.I. Project No. 1468

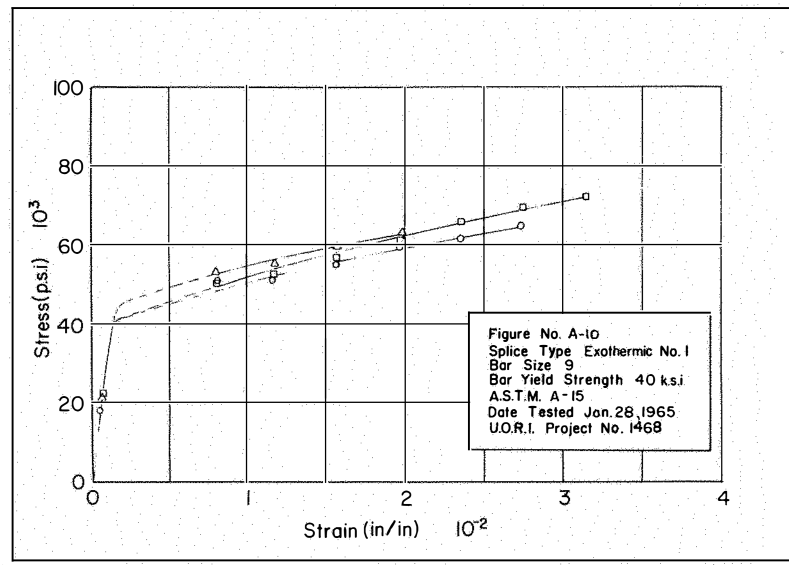

2 Samples
Theoretical Bar
Figure No. A-5
Splice Type Butt Weld
Bar Size 11
Bar Yield Strength 40 k.s.i
A.S.T.M. A-15
Date Tested Dec. 14, 1964
U.O.R.I. Project No. 1468
Stress (p.s.i) x 10^3
Strain (in/in) × 10^{-4}

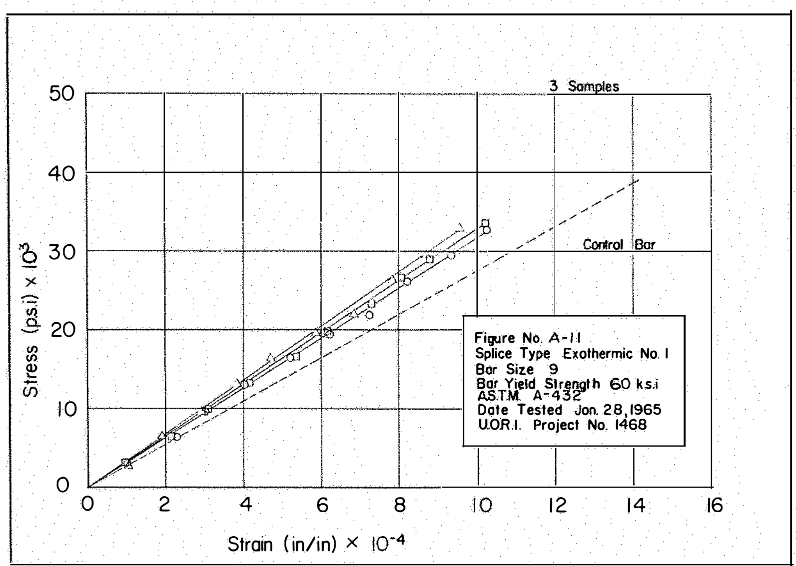

2 Samples
Theoretical Bar
Figure No. A-6
Splice Type Butt Weld
Bar Size 11
Bar Yield Strength 60 k.s.i
A.S.T.M. A-432
Date Tested Jan. 15, 1965
U.O.R.I. Project No. 1468
Stress (p.s.i) x 10^3
Strain (in/in) × 10^{-4}

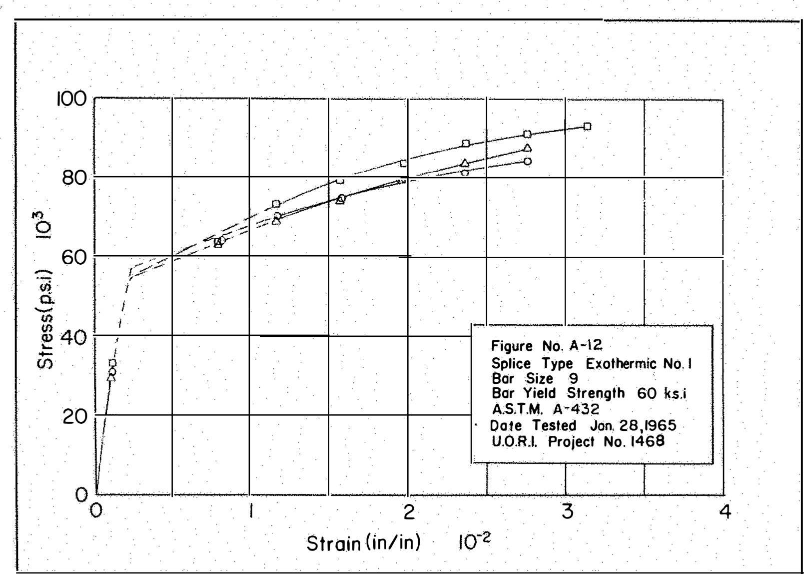

2 Samples
Theoretical Bar
Figure No. A-7
Splice Type Butt Weld
Bar Size 14S
Bar Yield Strength 40 k.s.i
A.S.T.M. A-408
Date Tested Dec. 14, 1964
U.O.R.I. Project No. 1468
Stress (p.s.i) x 10^3
Strain (in/in) × 10^{-4}

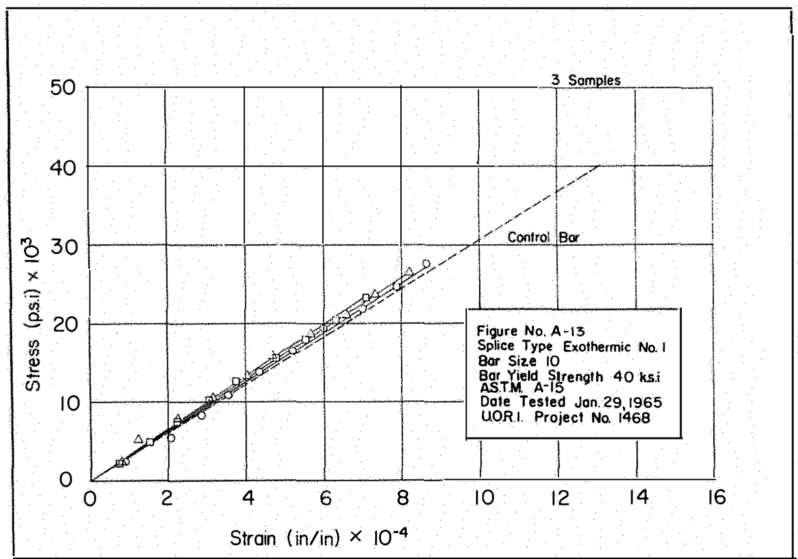

2 Samples
Theoretical Bar
Figure No. A-8
Splice Type Butt Weld
Bar Size 14S
Bar Yield Strength 60 k.s.i
A.S.T.M. A-432
Date Tested Dec. 14, 1964
U.O.R.I. Project No. 1468
Stress (p.s.i) x 10^3
Strain (in/in) × 10^{-4}


3 Samples
Control Bar
Stress (p.s.i) × 10³
Strain (in/in) × 10⁻⁴
Figure No. A-9
Splice Type Exothermic No. 1
Bar Size 9
Bar Yield Strength 40 k.s.i
A.S.T.M. A-15
Date Tested Jan. 28, 1965
U.O.R.I. Project No. 1468

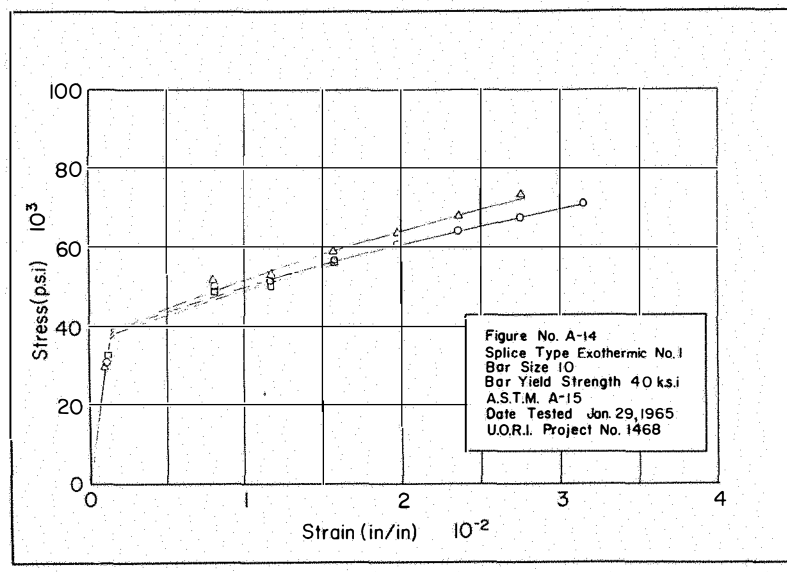

Stress (p.s.i) 10³
Strain (in/in) 10⁻²
Figure No. A-10
Splice Type Exothermic No. 1
Bar Size 9
Bar Yield Strength 40 k.s.i
A.S.T.M. A-15
Date Tested Jan. 28, 1965
U.O.R.I. Project No. 1468


3 Samples
Control Bar
Stress (p.s.i) × 10³
Strain (in/in) × 10⁻⁴
Figure No. A-11
Splice Type Exothermic No. 1
Bar Size 9
Bar Yield Strength 60 k.s.i
A.S.T.M. A-432
Date Tested Jan. 28, 1965
U.O.R.I. Project No. 1468

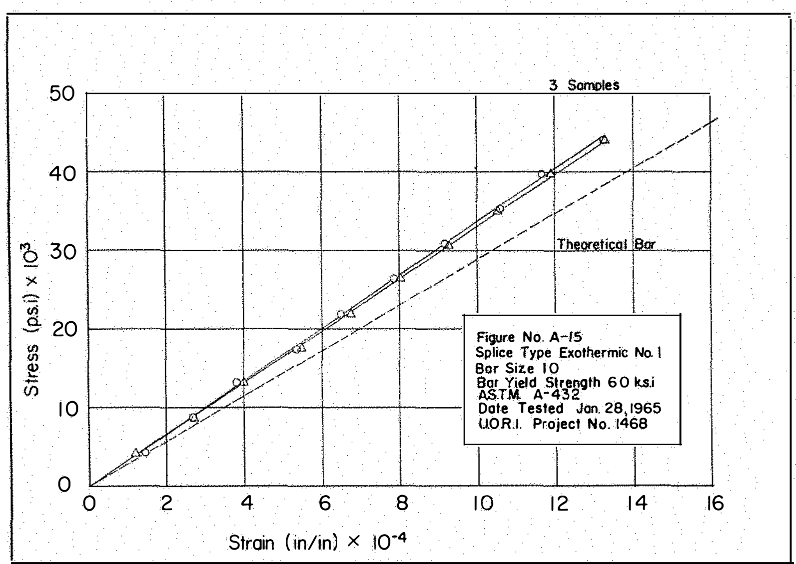

Stress (p.s.i) 10³
Strain (in/in) 10⁻²
Figure No. A-12
Splice Type Exothermic No. 1
Bar Size 9
Bar Yield Strength 60 k.s.i
A.S.T.M. A-432
Date Tested Jan. 28, 1965
U.O.R.I. Project No. 1468



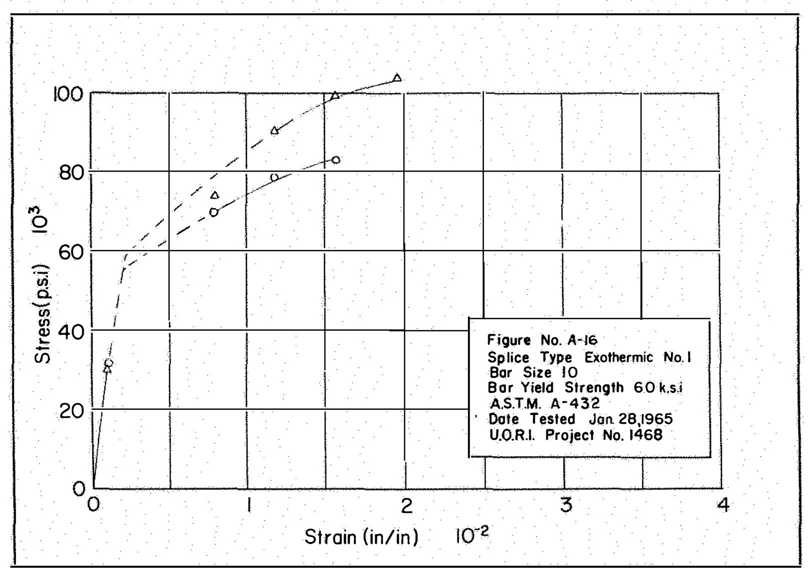
3 Samples
Control Bar
Stress (p.s.i) × 10^3
Strain (in/in) × 10^{-4}
Figure No. A-13
Splice Type Exothermic No. 1
Bar Size 10
Bar Yield Strength 40 ks.i
A.S.T.M. A-15
Date Tested Jan. 29, 1965
U.O.R.I. Project No. 1468



Stress (p.s.i) 10^3
Strain (in/in) 10^{-2}
Figure No. A-14
Splice Type Exothermic No. 1
Bar Size 10
Bar Yield Strength 40 ks.i
A.S.T.M. A-15
Date Tested Jan. 29, 1965
U.O.R.I. Project No. 1468



3 Samples
Theoretical Bar
Stress (p.s.i) × 10^3
Strain (in/in) × 10^{-4}
Figure No. A-15
Splice Type Exothermic No. 1
Bar Size 10
Bar Yield Strength 60 ks.i
A.S.T.M. A-432
Date Tested Jan. 28, 1965
U.O.R.I. Project No. 1468



Stress (p.s.i) 10^3
Strain (in/in) 10^{-2}
Figure No. A-16
Splice Type Exothermic No. 1
Bar Size 10
Bar Yield Strength 60 k.s.i
A.S.T.M. A-432
Date Tested Jan. 28, 1965
U.O.R.I. Project No. 1468

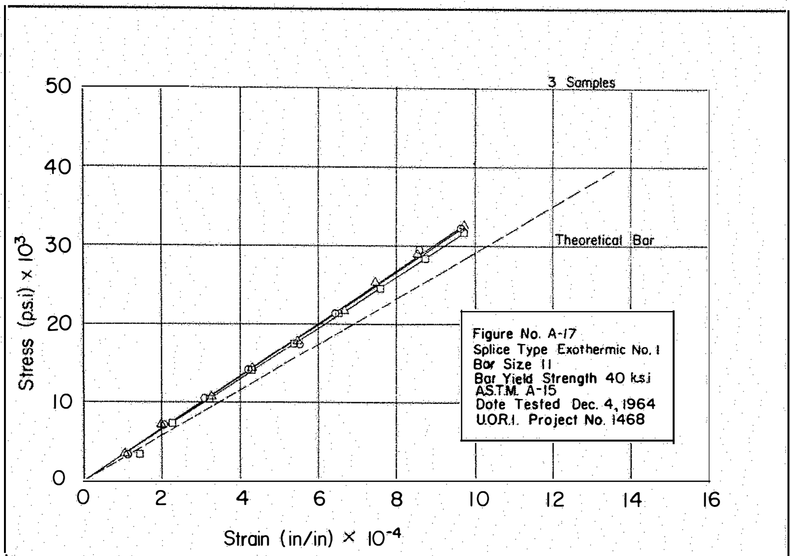


Figure No. A-17
Splice Type Exothermic No. I
Bar Size 11
Bar Yield Strength 40 k.s.i
A.S.T.M. A-15
Date Tested Dec. 4, 1964
U.O.R.I. Project No. 1468

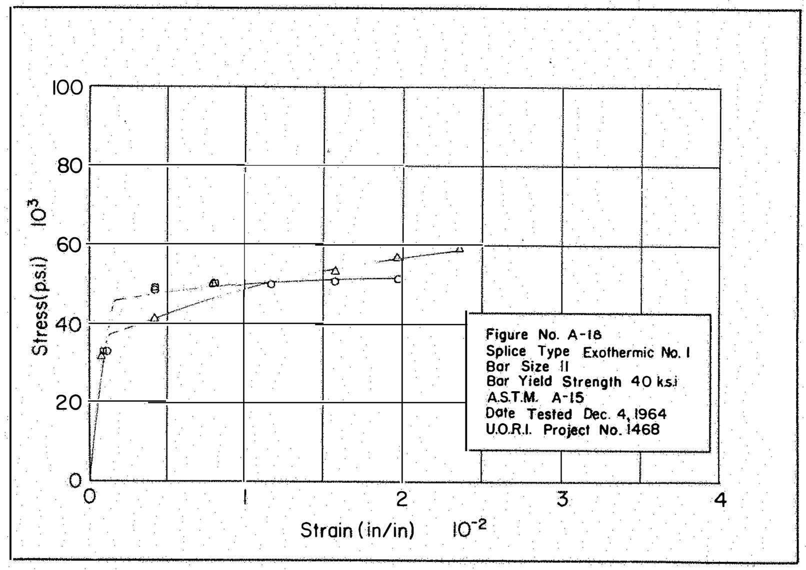


Figure No. A-18
Splice Type Exothermic No. I
Bar Size 11
Bar Yield Strength 40 k.s.i
A.S.T.M. A-15
Date Tested Dec. 4, 1964
U.O.R.I. Project No. 1468

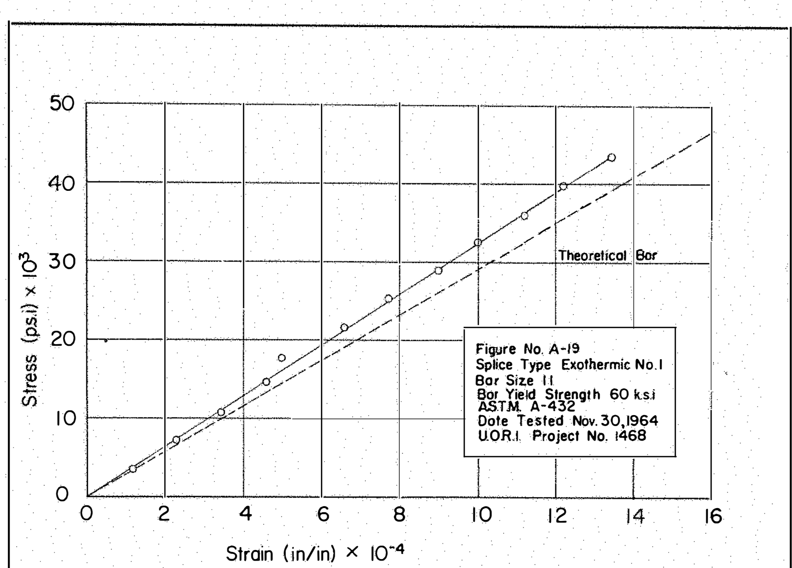


Figure No. A-19
Splice Type Exothermic No. I
Bar Size 11
Bar Yield Strength 60 k.s.i
A.S.T.M. A-432
Date Tested Nov. 30, 1964
U.O.R.I. Project No. 1468

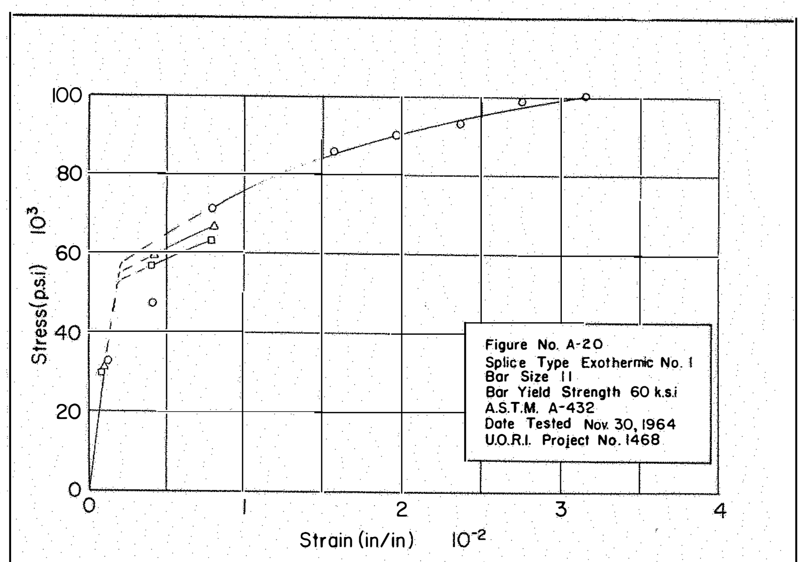
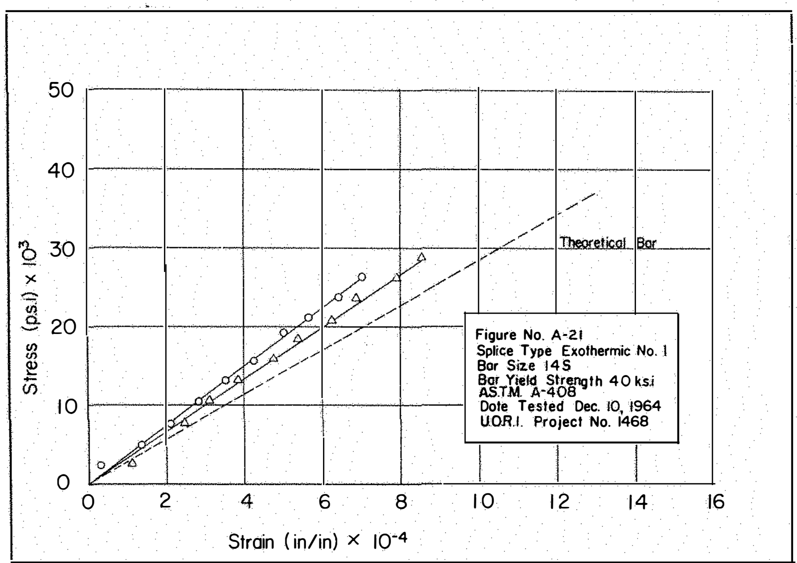
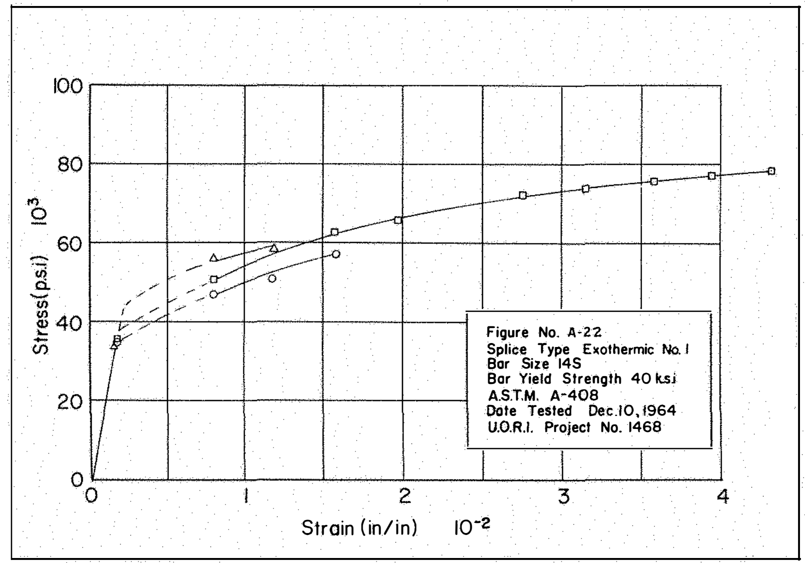


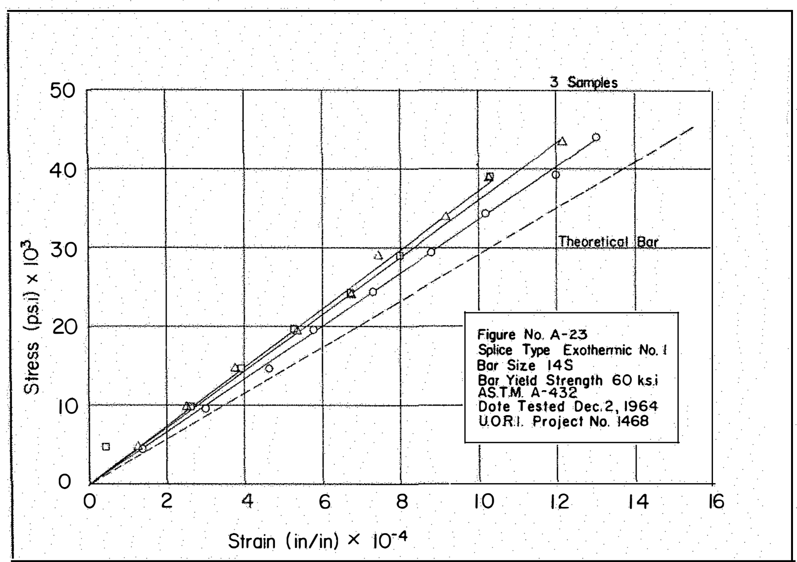
Figure No. A-20
Splice Type Exothermic No. 1
Bar Size 11
Bar Yield Strength 60 k.s.i
A.S.T.M. A-432
Date Tested Nov. 30, 1964
U.O.R.I. Project No. 1468



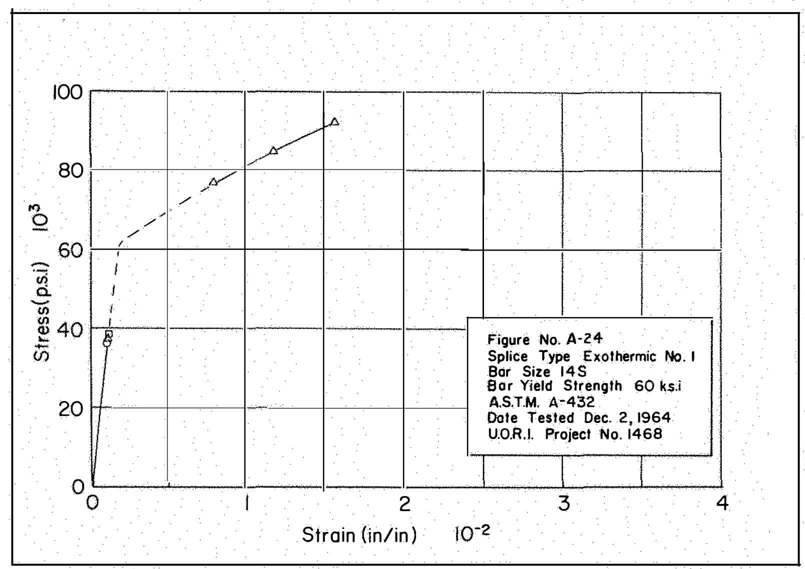
Stress (p.s.i) × 10^3
Strain (in/in) × 10^{-4}
Theoretical Bar
Figure No. A-21
Splice Type Exothermic No. 1
Bar Size 14S
Bar Yield Strength 40 ksi
A.S.T.M. A-408
Date Tested Dec. 10, 1964
U.O.R.I. Project No. 1468



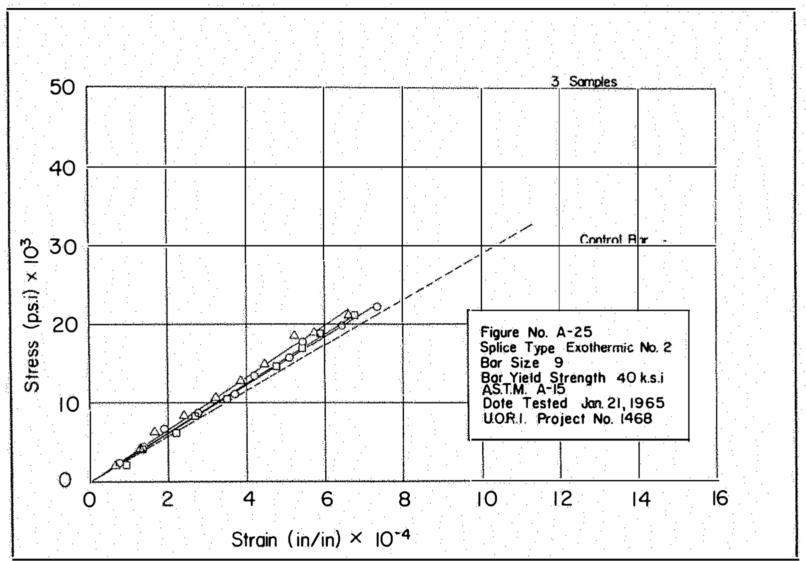
Stress(p.s.i) 10^3
Strain (in/in) 10^{-2}
Figure No. A-22
Splice Type Exothermic No. 1
Bar Size 14S
Bar Yield Strength 40 ksi
A.S.T.M. A-408
Date Tested Dec. 10, 1964
U.O.R.I. Project No. 1468

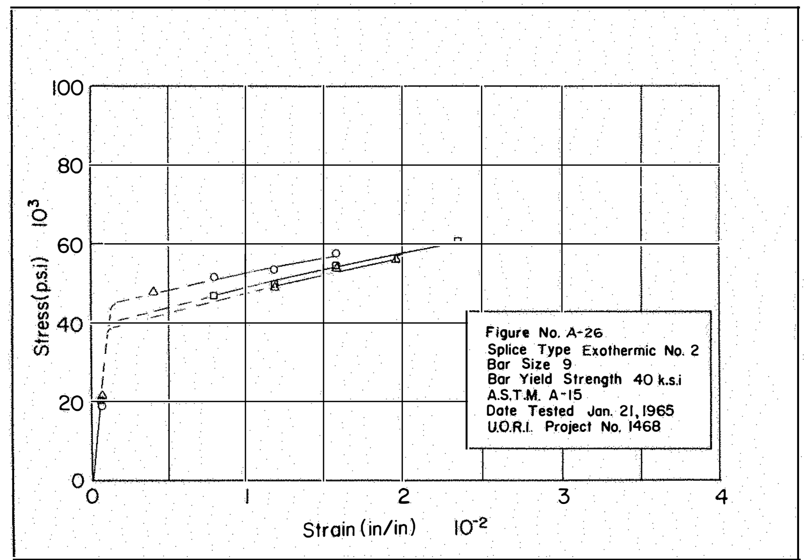


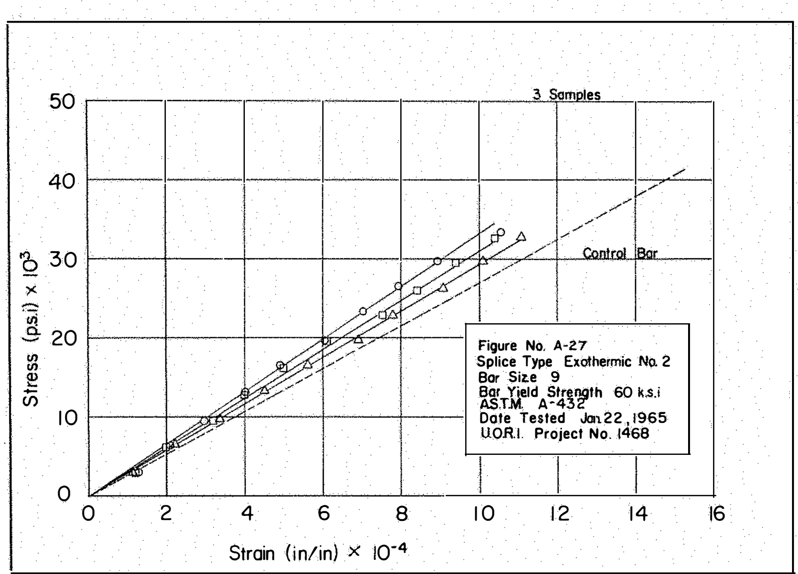
3 Samples
Stress (p.s.i) × 10^3
Strain (in/in) × 10^{-4}
Theoretical Bar
Figure No. A-23
Splice Type Exothermic No. 1
Bar Size 14S
Bar Yield Strength 60 ksi
A.S.T.M. A-432
Date Tested Dec. 2, 1964
U.O.R.I. Project No. 1468

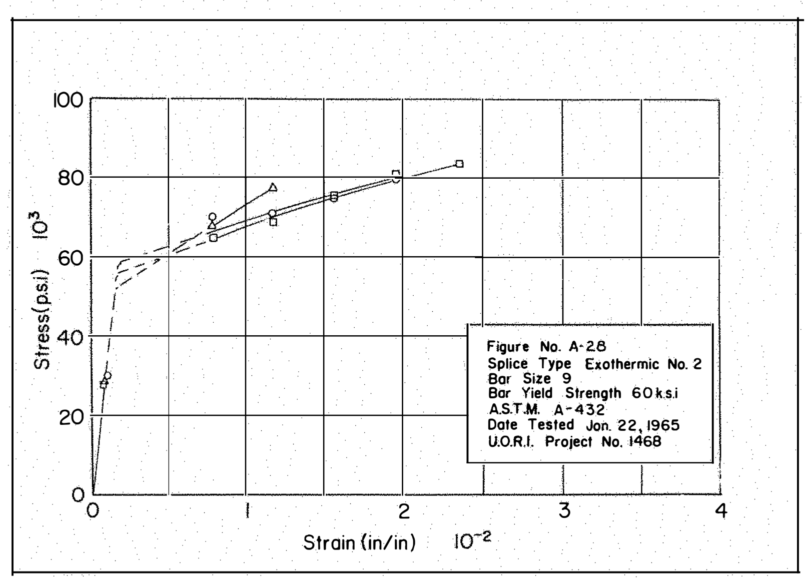


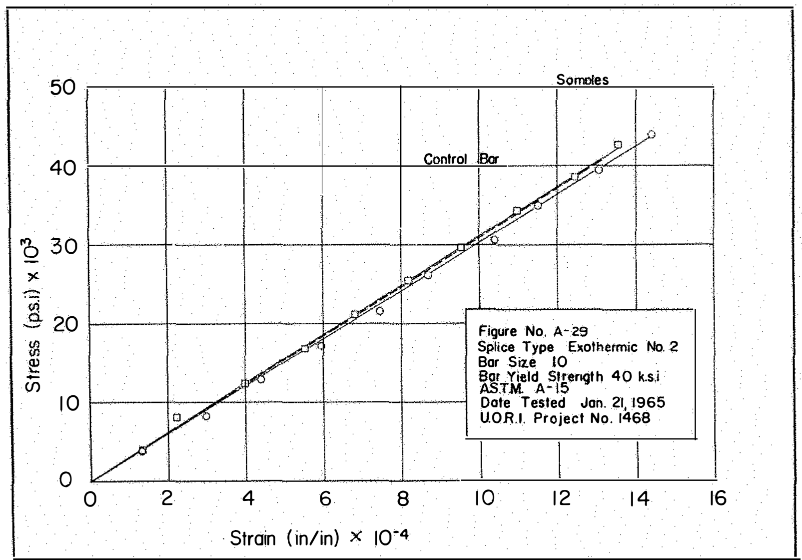
Stress(p.s.i) 10^3
Strain (in/in) 10^{-2}
Figure No. A-24
Splice Type Exothermic No. 1
Bar Size 14S
Bar Yield Strength 60 ksi
A.S.T.M. A-432
Date Tested Dec. 2, 1964
U.O.R.I. Project No. 1468

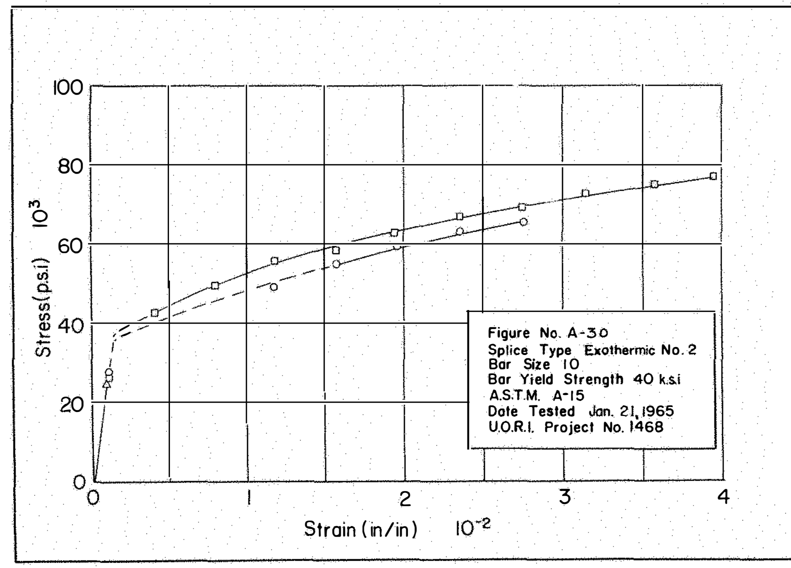

3 Samples
Control Bar
Stress (p.s.i) × 10^3
Strain (in/in) × 10^{-4}
Figure No. A-25
Splice Type Exothermic No. 2
Bar Size 9
Bar Yield Strength 40 k.s.i
A.S.T.M. A-15
Date Tested Jan. 21, 1965
U.O.R.I. Project No. 1468

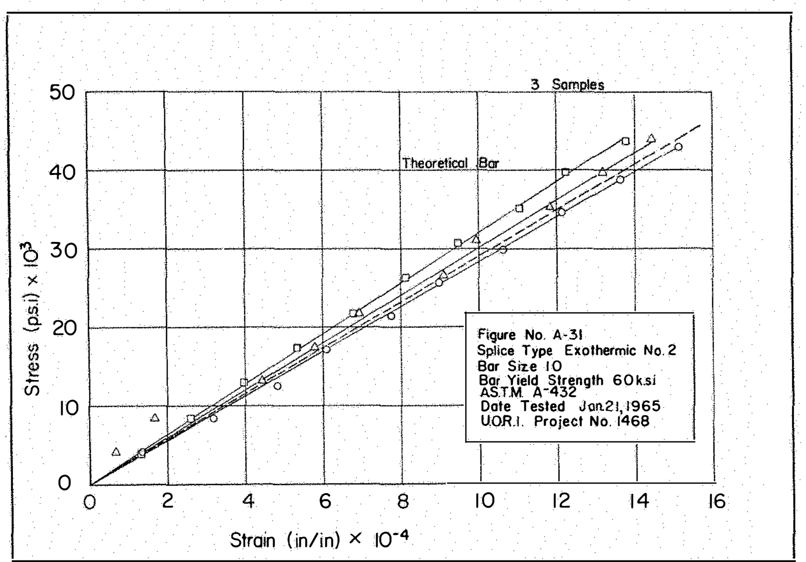

Stress(p.s.i) 10^3
Strain (in/in) 10^{-2}
Figure No. A-26
Splice Type Exothermic No. 2
Bar Size 9
Bar Yield Strength 40 k.s.i
A.S.T.M. A-15
Date Tested Jan. 21, 1965
U.O.R.I. Project No. 1468

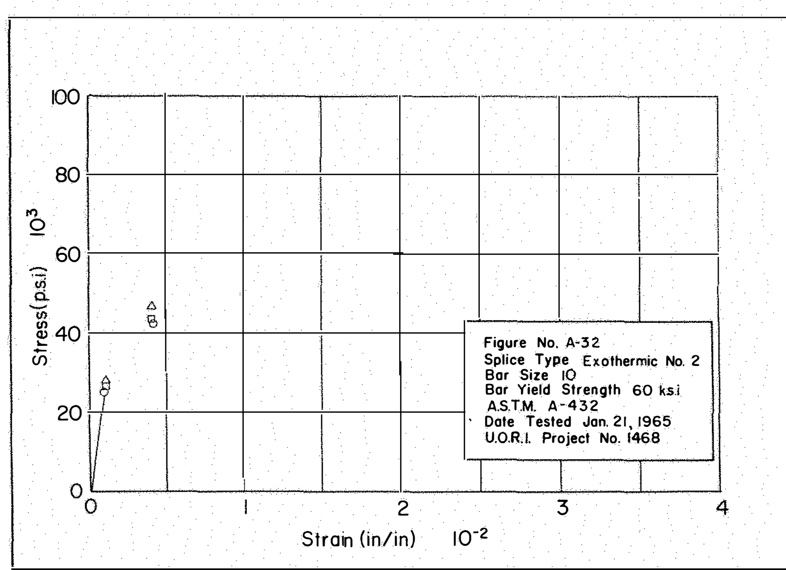

3 Samples
Control Bar
Stress (p.s.i) × 10^3
Strain (in/in) × 10^{-4}
Figure No. A-27
Splice Type Exothermic No. 2
Bar Size 9
Bar Yield Strength 60 k.s.i
A.S.T.M. A-432
Date Tested Jan. 22, 1965
U.O.R.I. Project No. 1468

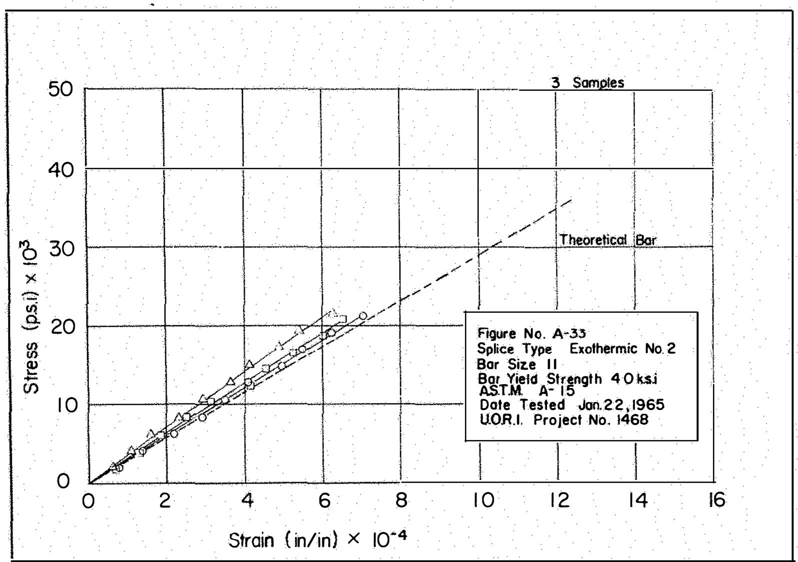

Stress(p.s.i) 10^3
Strain (in/in) 10^{-2}
Figure No. A-28
Splice Type Exothermic No. 2
Bar Size 9
Bar Yield Strength 60 k.s.i
A.S.T.M. A-432
Date Tested Jan. 22, 1965
U.O.R.I. Project No. 1468

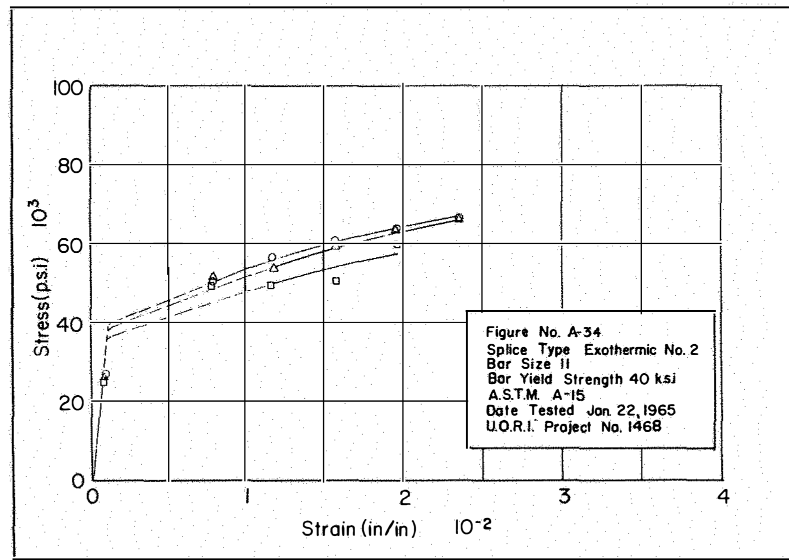

Samples
Control Bar
Stress (ps.i) x 10³
Strain (in/in) × 10⁻⁴
Figure No. A-29
Splice Type Exothermic No. 2
Bar Size 10
Bar Yield Strength 40 k.s.i
A.S.T.M. A-15
Date Tested Jan. 21, 1965
U.O.R.I. Project No. 1468

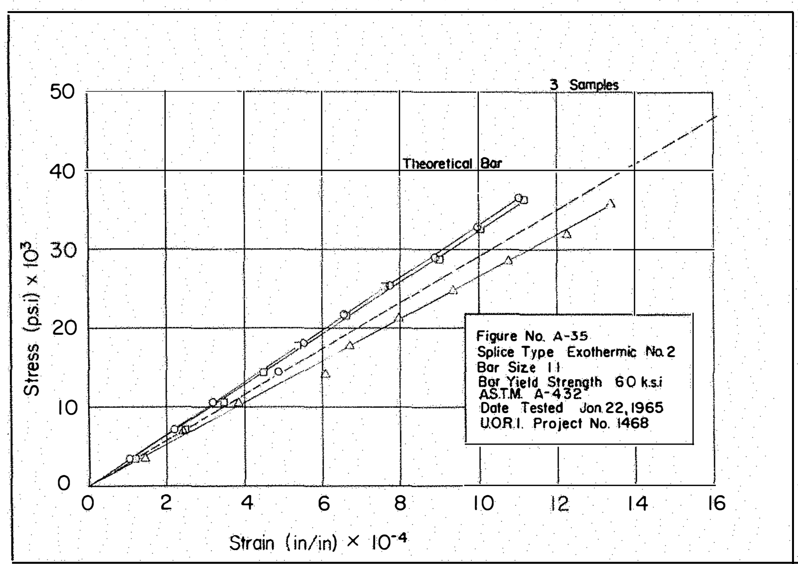

Stress (p.s.i) 10³
Strain (in/in) 10⁻²
Figure No. A-30
Splice Type Exothermic No. 2
Bar Size 10
Bar Yield Strength 40 k.s.i
A.S.T.M. A-15
Date Tested Jan. 21, 1965
U.O.R.I. Project No. 1468

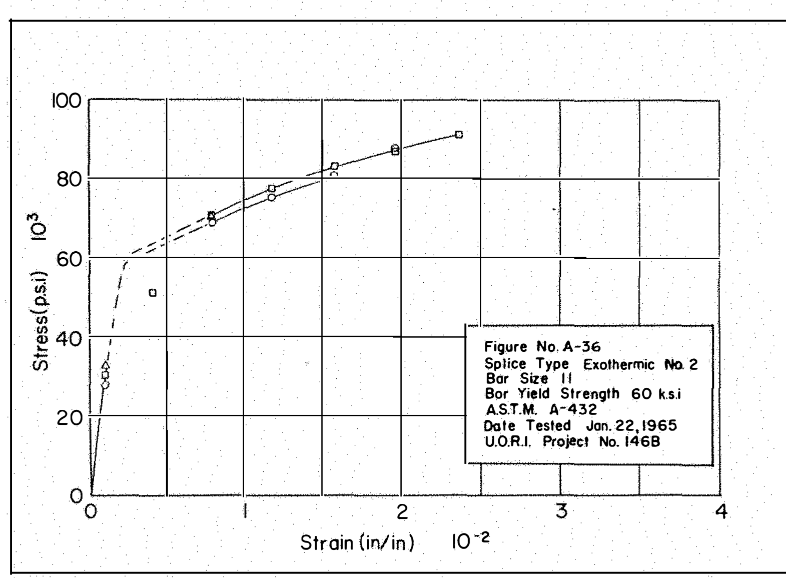

3 Samples
Theoretical Bar
Stress (ps.i) x 10³
Strain (in/in) × 10⁻⁴
Figure No. A-31
Splice Type Exothermic No. 2
Bar Size 10
Bar Yield Strength 60 k.s.i
A.S.T.M. A-432
Date Tested Jan. 21, 1965
U.O.R.I. Project No. 1468

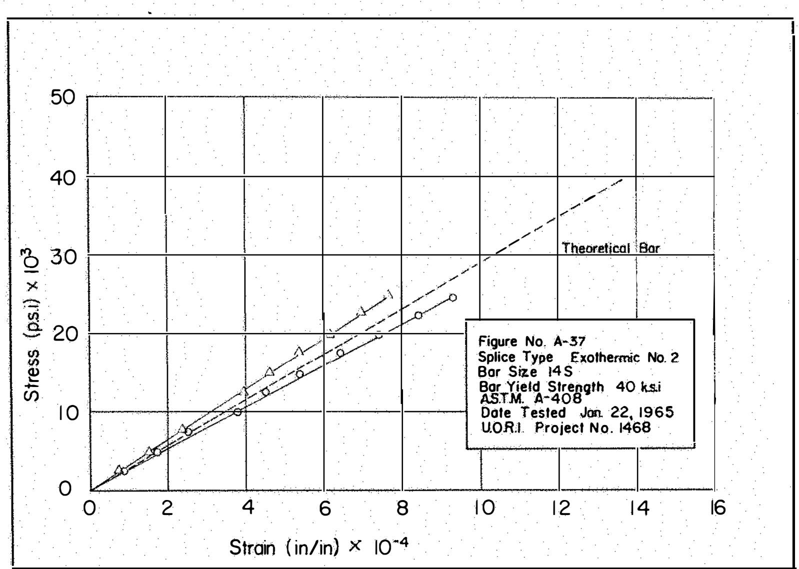

Stress (p.s.i) 10³
Strain (in/in) 10⁻²
Figure No. A-32
Splice Type Exothermic No. 2
Bar Size 10
Bar Yield Strength 60 k.s.i
A.S.T.M. A-432
Date Tested Jan. 21, 1965
U.O.R.I. Project No. 1468


3 Samples
Theoretical Bar
Stress (p.s.i) x 10^3
Strain (in/in) × 10^-4
Figure No. A-33
Splice Type Exothermic No. 2
Bar Size 11
Bar Yield Strength 40 k.s.i
A.S.T.M. A-15
Date Tested Jan. 22, 1965
U.O.R.I. Project No. 1468


Stress (p.s.i) 10^3
Strain (in/in) 10^-2
Figure No. A-34
Splice Type Exothermic No. 2
Bar Size 11
Bar Yield Strength 40 k.s.i
A.S.T.M. A-15
Date Tested Jan. 22, 1965
U.O.R.I. Project No. 1468


3 Samples
Theoretical Bar
Stress (p.s.i) x 10^3
Strain (in/in) × 10^-4
Figure No. A-35
Splice Type Exothermic No. 2
Bar Size 11
Bar Yield Strength 60 k.s.i
A.S.T.M. A-432
Date Tested Jan. 22, 1965
U.O.R.I. Project No. 1468


Stress (p.s.i) 10^3
Strain (in/in) 10^-2
Figure No. A-36
Splice Type Exothermic No. 2
Bar Size 11
Bar Yield Strength 60 k.s.i
A.S.T.M. A-432
Date Tested Jan. 22, 1965
U.O.R.I. Project No. 1468



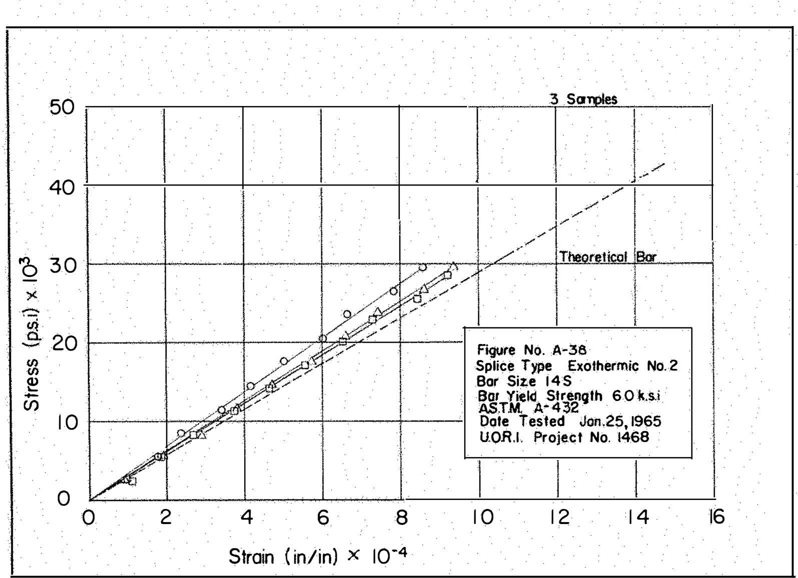
EXOTHERMIC NO. 2

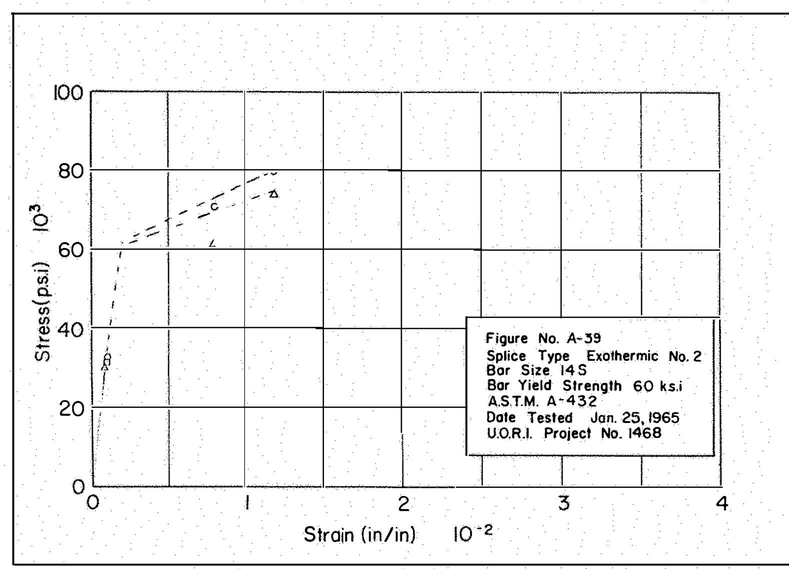
BAR SIZE - 14S

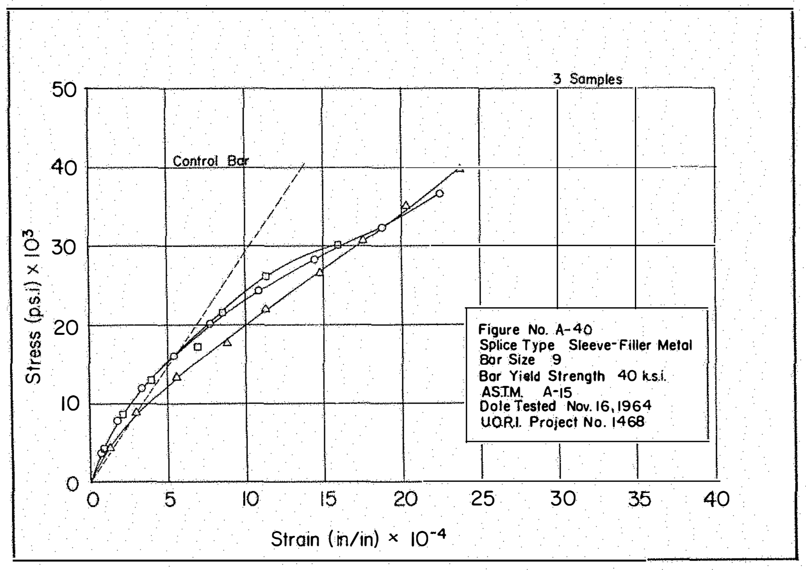
BAR YIELD STRENGTH - 40KSI

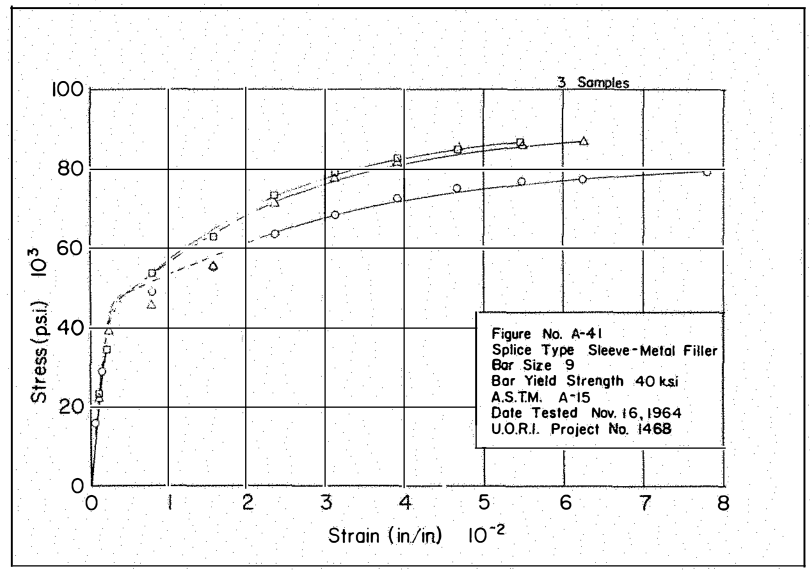
ASTM - A408

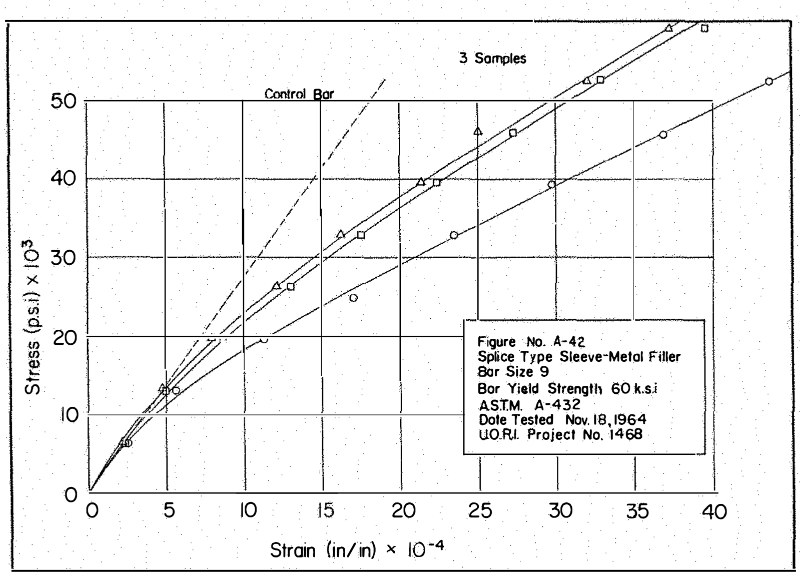
(FAILED BELOW NOMINAL YIELD STRENGTH)

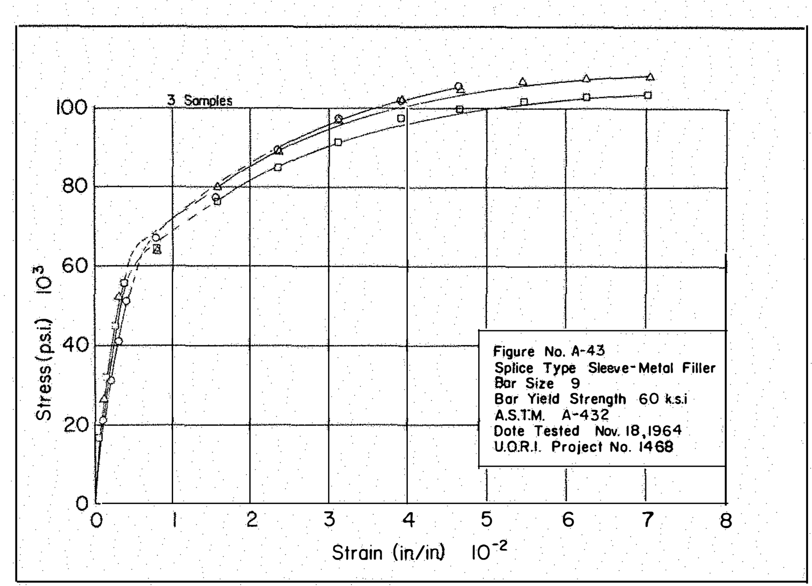





3 Samples
Control Bar
Stress (p.s.i) × 10^3
Strain (in/in) × 10^{-4}
Figure No. A-40
Splice Type Sleeve-Filler Metal
Bar Size 9
Bar Yield Strength 40 k.s.i.
A.S.T.M. A-15
Date Tested Nov. 16, 1964
U.O.R.I. Project No. 1468


3 Samples
Stress (p.s.i) 10^3
Strain (in/in) 10^{-2}
Figure No. A-41
Splice Type Sleeve-Metal Filler
Bar Size 9
Bar Yield Strength 40 k.s.i
A.S.T.M. A-15
Date Tested Nov. 16, 1964
U.O.R.I. Project No. 1468


3 Samples
Control Bar
Stress (p.s.i) × 10^3
Strain (in/in) × 10^{-4}
Figure No. A-42
Splice Type Sleeve-Metal Filler
Bar Size 9
Bar Yield Strength 60 k.s.i
A.S.T.M. A-432
Date Tested Nov. 18, 1964
U.O.R.I. Project No. 1468


3 Samples
Stress (p.s.i.) 10^3
Strain (in/in) 10^{-2}
Figure No. A-43
Splice Type Sleeve-Metal Filler
Bar Size 9
Bar Yield Strength 60 k.s.i
A.S.T.M. A-432
Date Tested Nov. 18, 1964
U.O.R.I. Project No. 1468

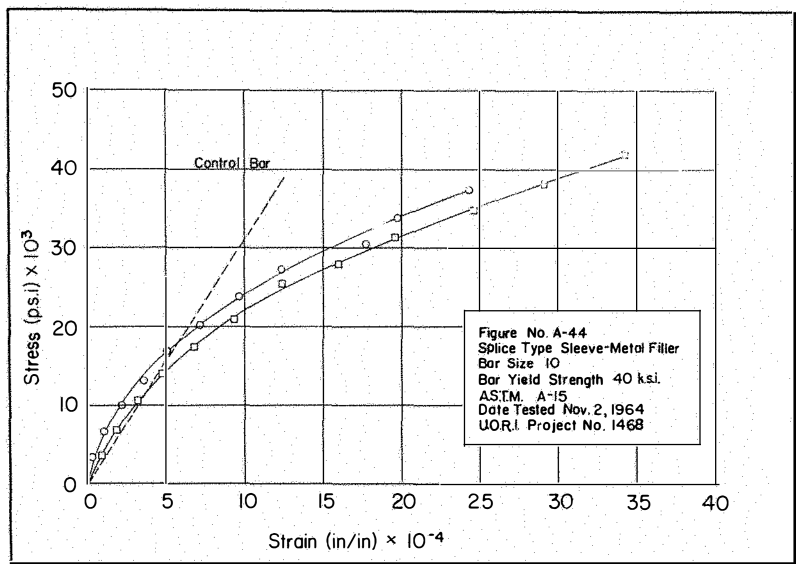


Figure No. A-44
Splice Type Sleeve-Metal Filler
Bar Size 10
Bar Yield Strength 40 k.s.i.
A.S.T.M. A-15
Date Tested Nov. 2, 1964
U.O.R.I. Project No. 1468

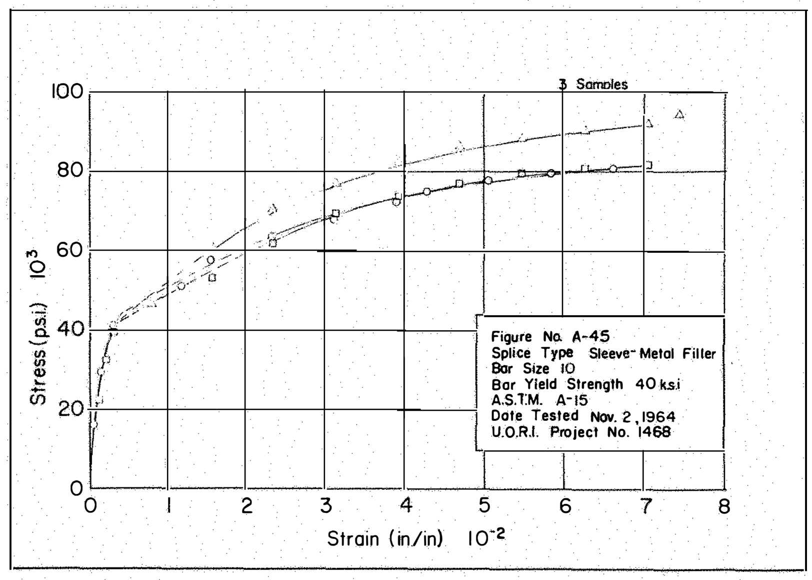


Figure No. A-45
Splice Type Sleeve-Metal Filler
Bar Size 10
Bar Yield Strength 40 k.s.i
A.S.T.M. A-15
Date Tested Nov. 2, 1964
U.O.R.I. Project No. 1468

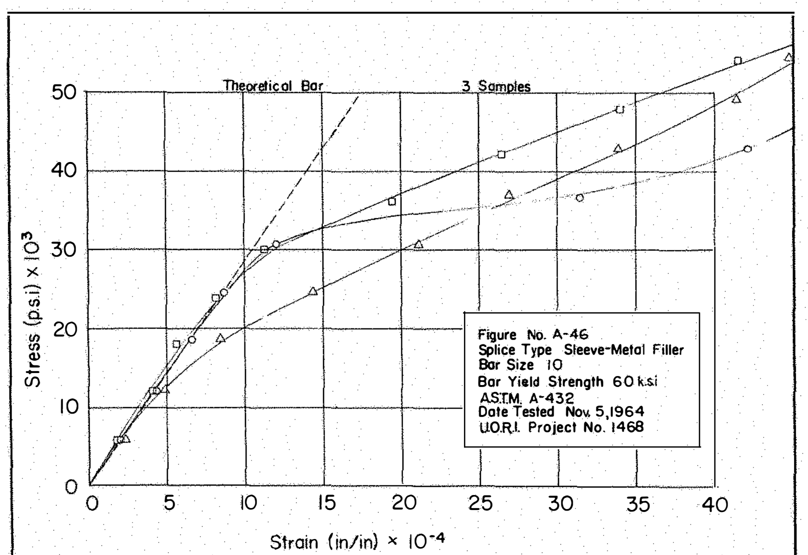


Figure No. A-46
Splice Type Sleeve-Metal Filler
Bar Size 10
Bar Yield Strength 60 k.s.i
A.S.T.M. A-432
Date Tested Nov. 5, 1964
U.O.R.I. Project No. 1468

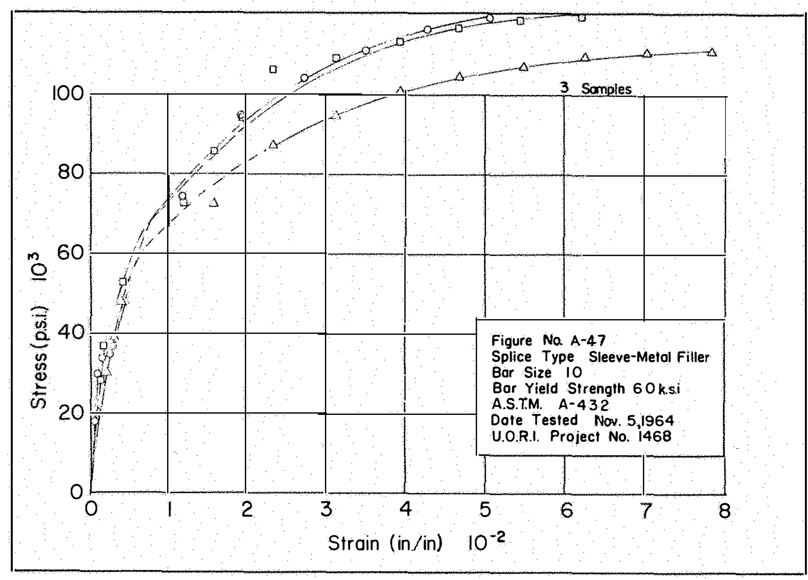


Figure No. A-47
Splice Type Sleeve-Metal Filler
Bar Size 10
Bar Yield Strength 60 k.s.i
A.S.T.M. A-432
Date Tested Nov. 5, 1964
U.O.R.I. Project No. 1468

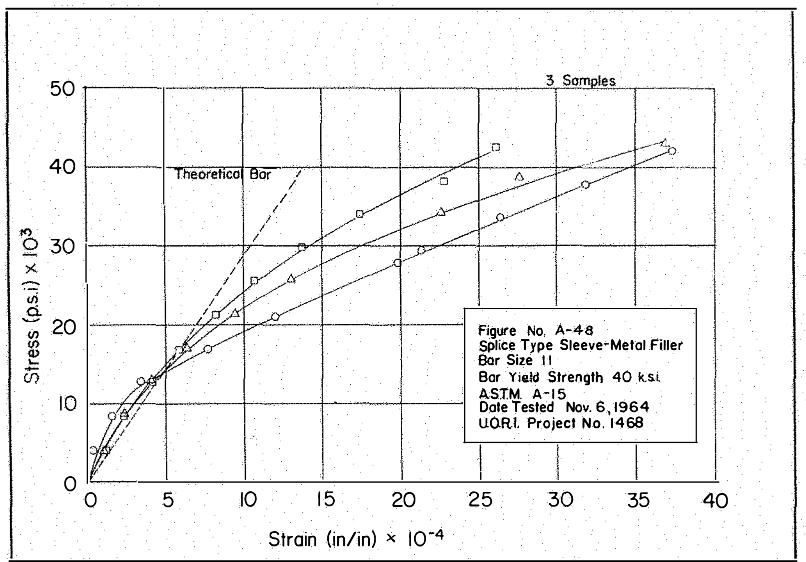


Figure No. A-48
Splice Type Sleeve-Metal Filler
Bar Size 11
Bar Yield Strength 40 k.s.i.
A.S.T.M. A-15
Date Tested Nov. 6, 1964
U.O.R.I. Project No. 1468

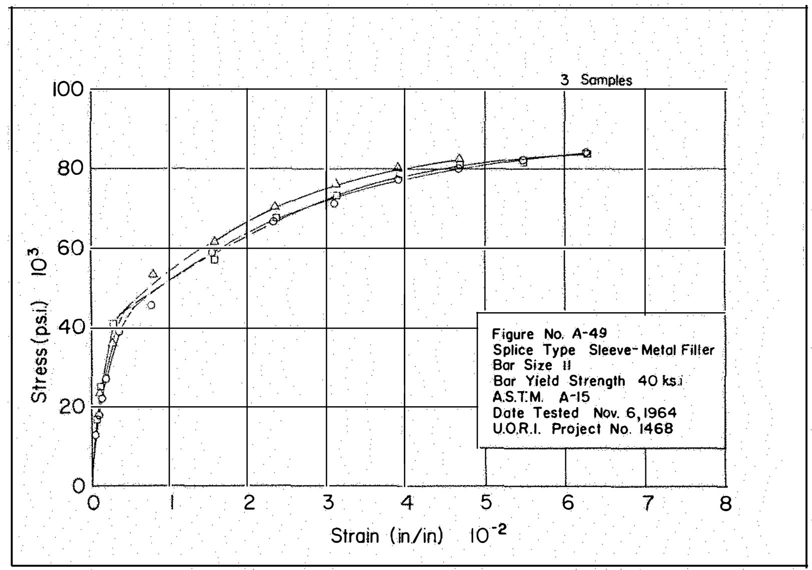


Figure No. A-49
Splice Type Sleeve-Metal Filler
Bar Size 11
Bar Yield Strength 40 ks.i
A.S.T.M. A-15
Date Tested Nov. 6, 1964
U.O.R.I. Project No. 1468

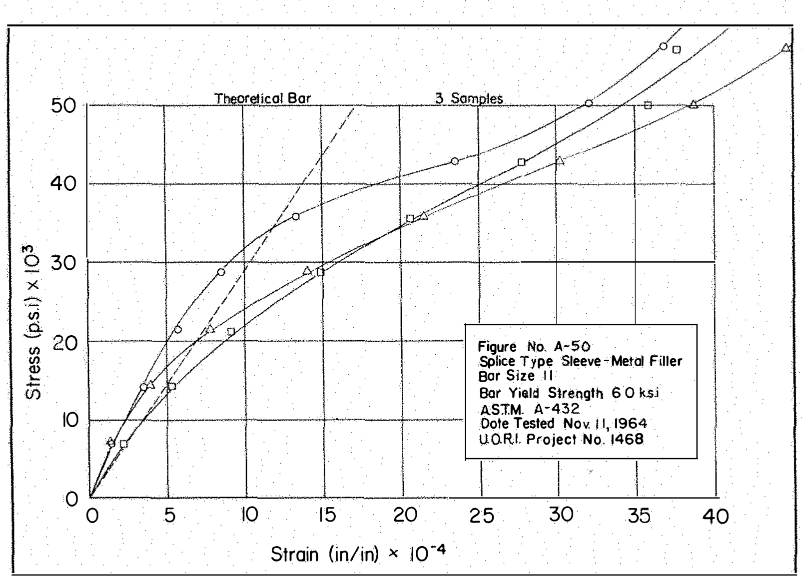


Figure No. A-50
Splice Type Sleeve-Metal Filler
Bar Size 11
Bar Yield Strength 60 ks.i
A.S.T.M. A-432
Date Tested Nov. 11, 1964
U.O.R.I. Project No. 1468

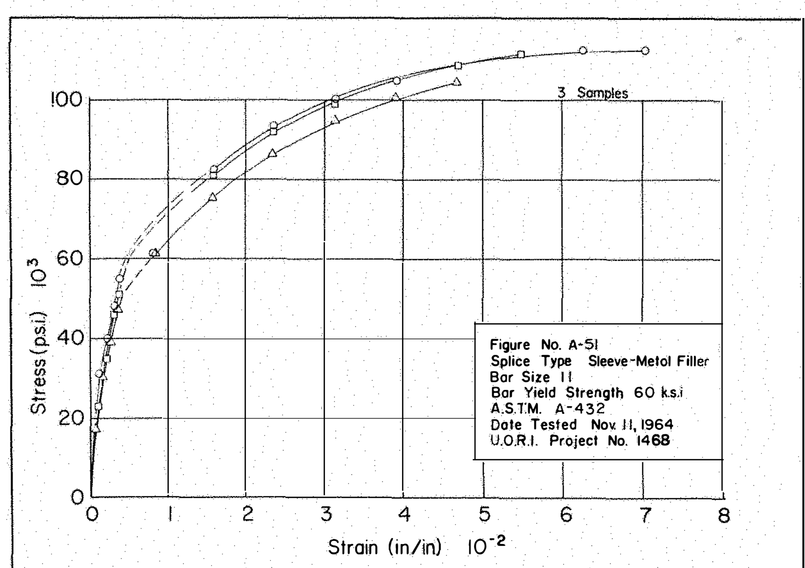
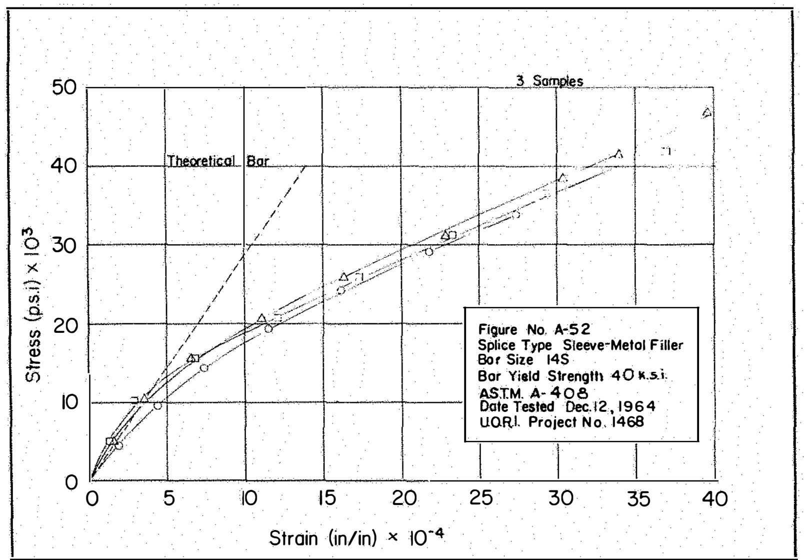
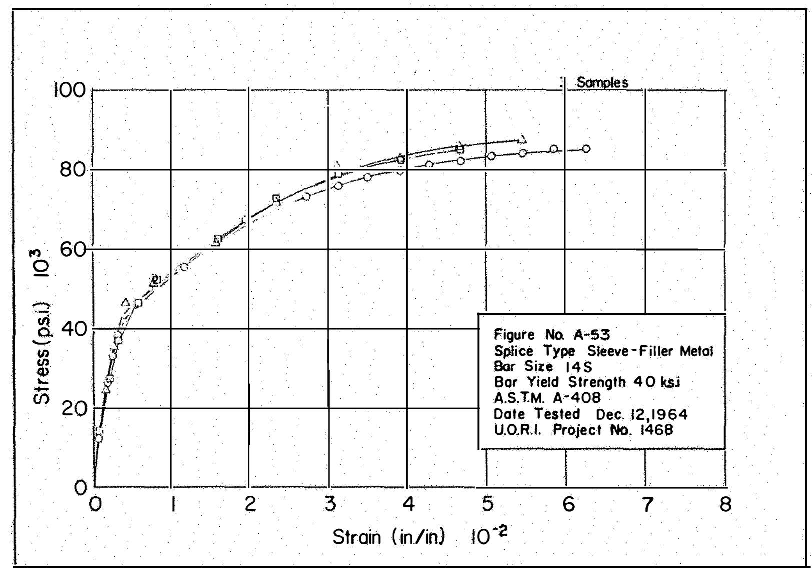
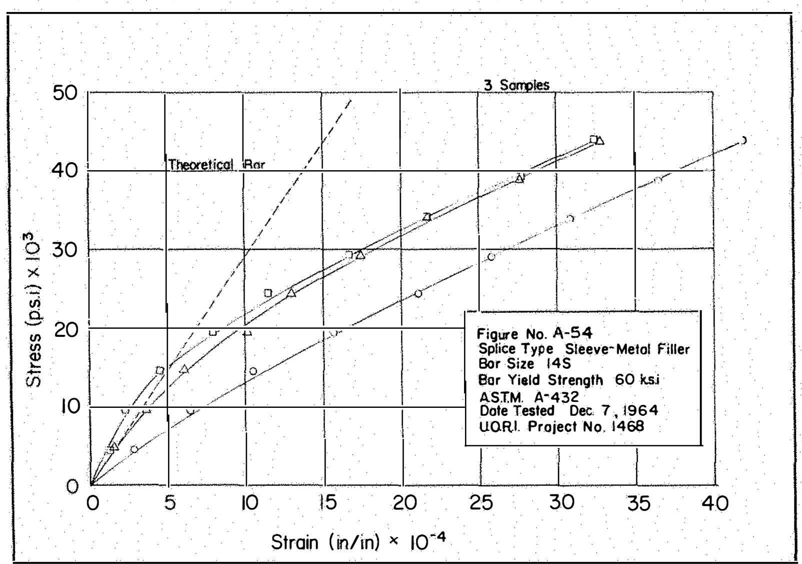
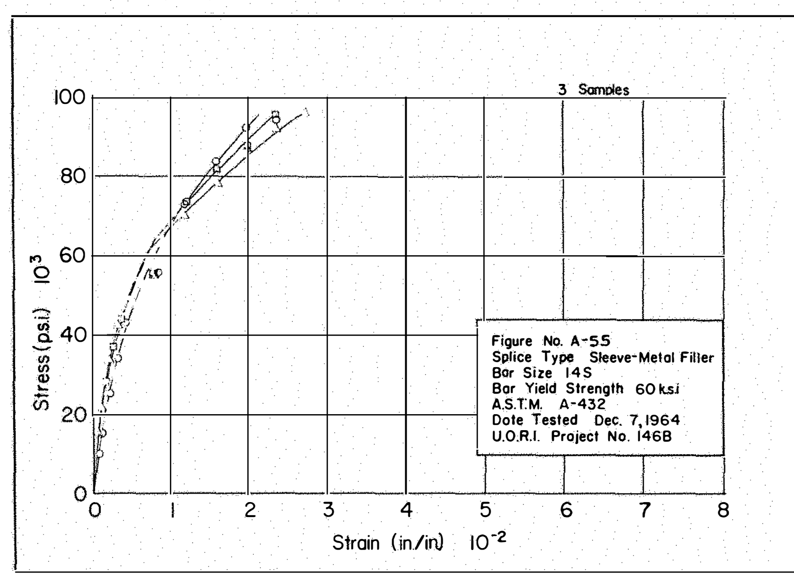


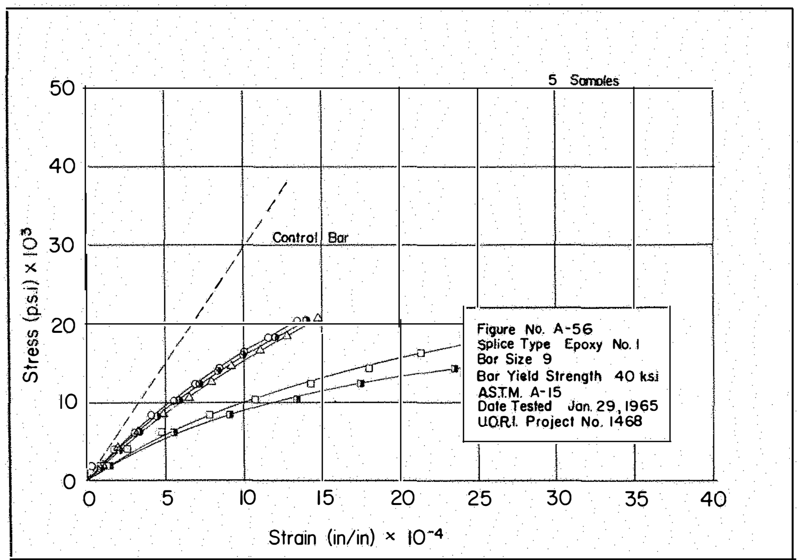
Figure No. A-51
Splice Type Sleeve-Metal Filler
Bar Size 11
Bar Yield Strength 60 ks.i
A.S.T.M. A-432
Date Tested Nov. 11, 1964
U.O.R.I. Project No. 1468

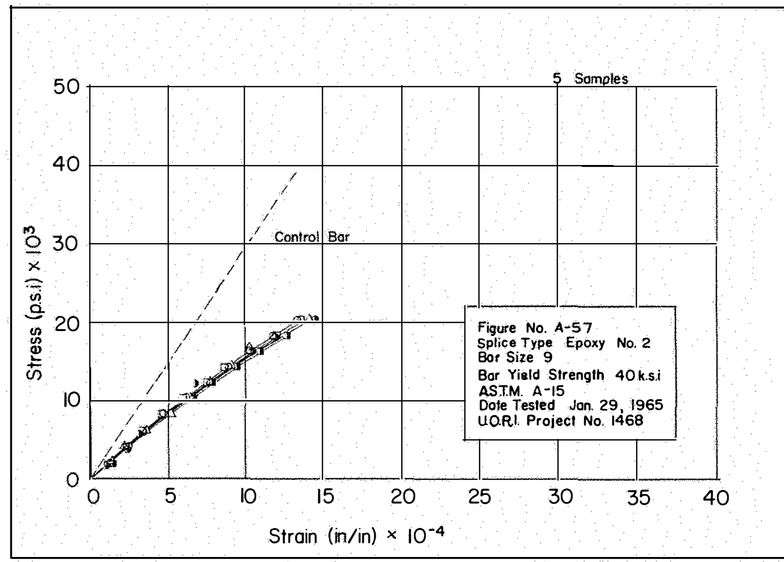

3 Samples
Theoretical Bar
Stress (p.s.i) x 10³
Strain (in/in) × 10⁻⁴
Figure No. A-52
Splice Type Sleeve-Metal Filler
Bar Size 14S
Bar Yield Strength 40 k.s.i.
A.S.T.M. A-408
Date Tested Dec.12, 1964
U.O.R.I. Project No. 1468


3 Samples
Stress (p.s.i) 10³
Strain (in/in) 10⁻²
Figure No. A-53
Splice Type Sleeve-Filler Metal
Bar Size 14S
Bar Yield Strength 40 ksi
A.S.T.M. A-408
Date Tested Dec. 12, 1964
U.O.R.I. Project No. 1468


3 Samples
Theoretical Bar
Stress (p.s.i) x 10³
Strain (in/in) × 10⁻⁴
Figure No. A-54
Splice Type Sleeve-Metal Filler
Bar Size 14S
Bar Yield Strength 60 ksi
A.S.T.M. A-432
Date Tested Dec. 7, 1964
U.O.R.I. Project No. 1468


3 Samples
Stress (p.s.i) 10³
Strain (in/in) 10⁻²
Figure No. A-55
Splice Type Sleeve-Metal Filler
Bar Size 14S
Bar Yield Strength 60 ksi
A.S.T.M. A-432
Date Tested Dec. 7, 1964
U.O.R.I. Project No. 1468





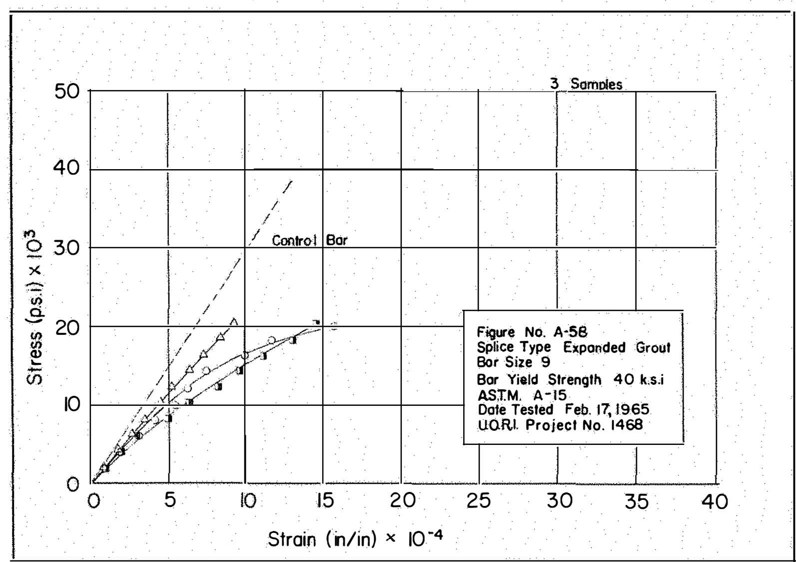
EPOXY SPLICE NO. 3

BAR SIZE - 9

BAR YIELD STRENGTH - 40KSI

ASTM - A15

(NO DATA TAKEN - FAILED PREMATURELY)



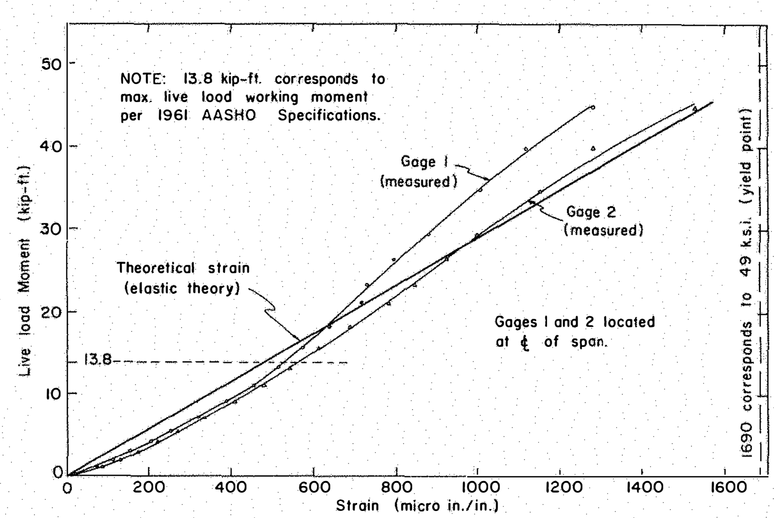


FIGURE A-59. STEEL STRAINS AT MID-SPAN — BEAM NO. 1 (CONTROL)

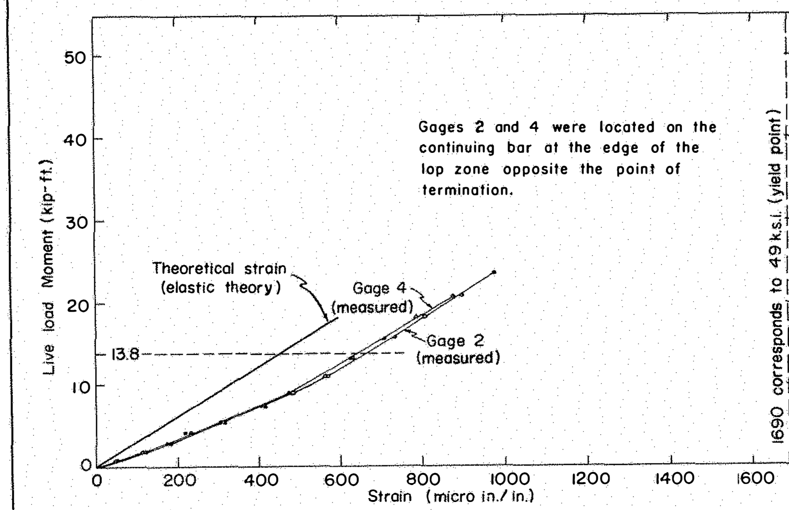


FIGURE A-60. STEEL STRAINS AT APPROX. MID-SPAN — BEAM NO. 2 (LAP 25 D)

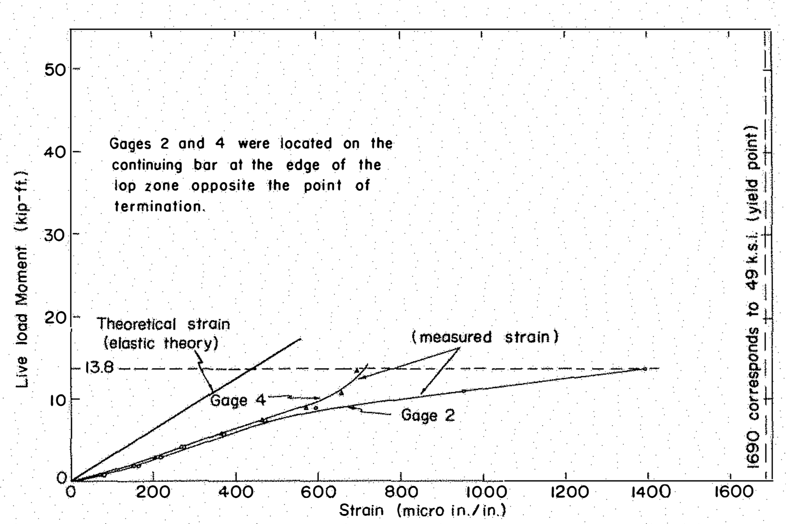


FIGURE A-61. STEEL STRAINS AT APPROX. MID-SPAN — BEAM NO. 3 (17 D LAP)

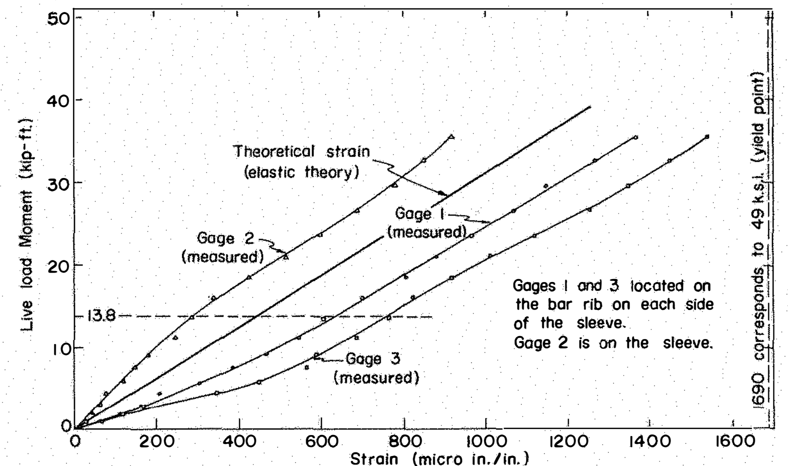


FIGURE A-62. STEEL STRAINS AT MID-SPAN
BEAM NO. 4 (SLEEVE WITH METAL FILLER)

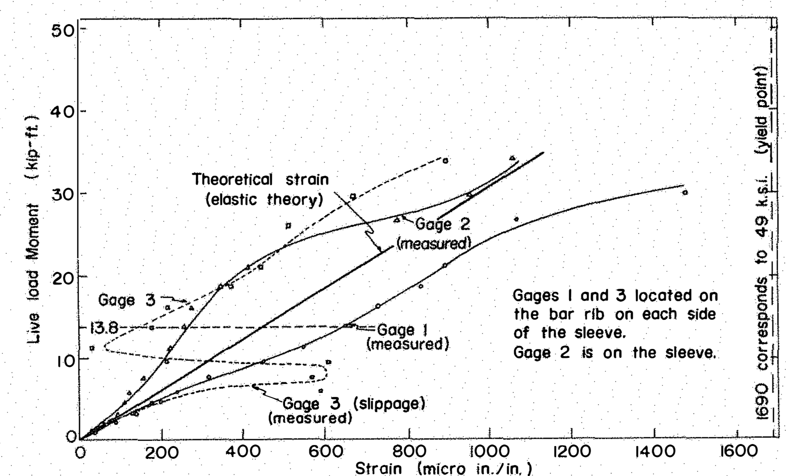


FIGURE A-63. STEEL STRAINS AT MID-SPAN
BEAM NO. 5 (SLEEVE WITH METAL FILLER)

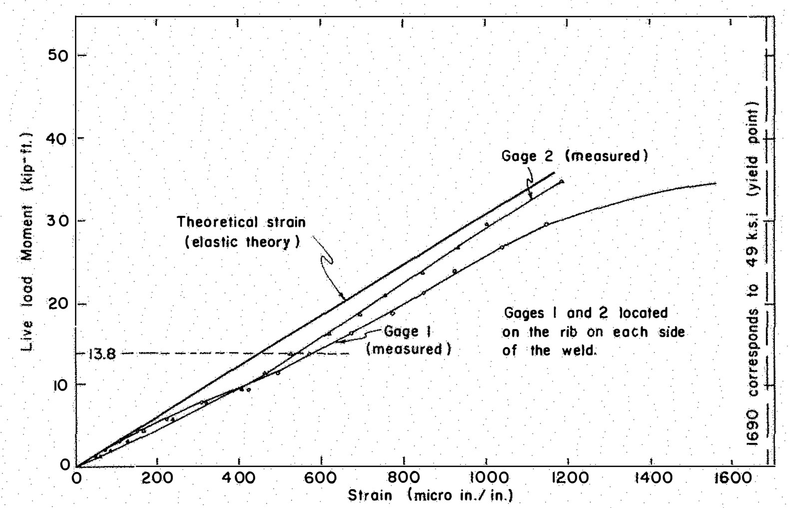


FIGURE A 64. STEEL STRAINS AT MID SPAN——BEAM NO. 6 (EXOTHERMIC #1)

FIGURE A-65. STRESS-STRAIN CURVE FOR NO. 11, ASTM A-15, REINFORCING BAR USED FOR FLEXURAL TESTS.